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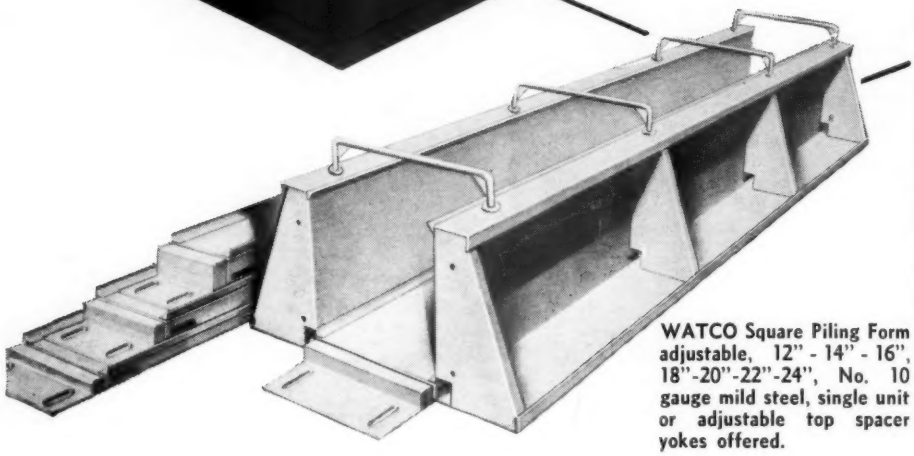

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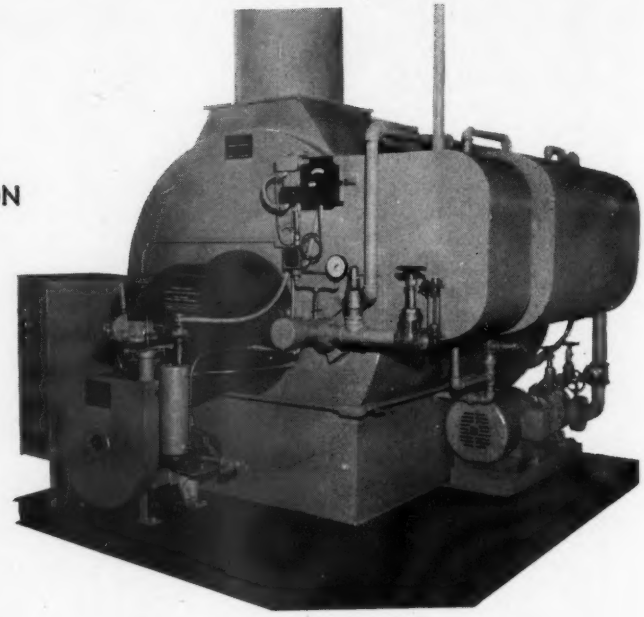

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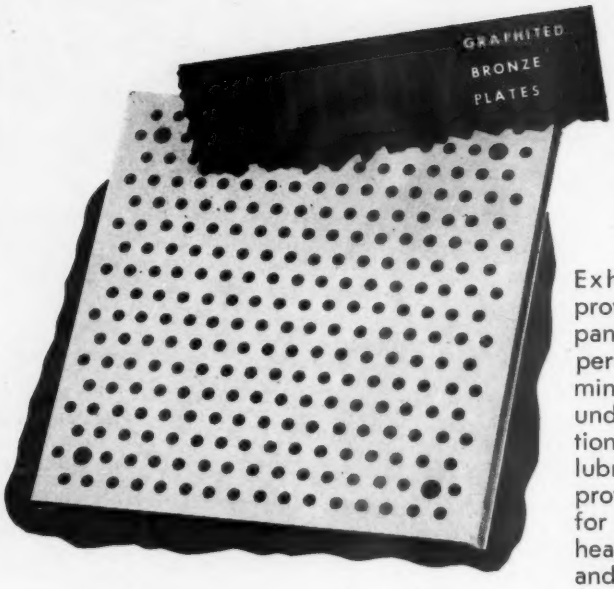

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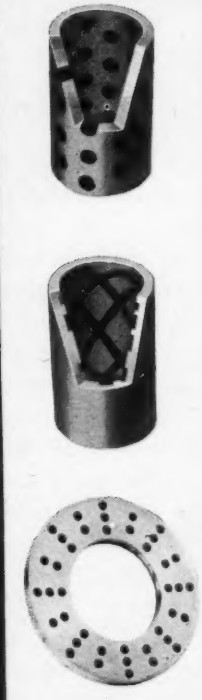

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From the editor . . .

The World Conference in San Francisco was a great success. All those who attended met and heard the world's most distinguished engineers, architects, builders and producers in the prestressed concrete industry. The Conference was superbly arranged and the program was second to none. Almost 1200 registrants from some 27 countries made the Conference a great success. Congratulations to the Arrangements Committee and to all who contributed to its tremendous achievement.

The new PCI Officers and Directors, elected during the Convention in San Francisco, are all well known to you. They need your support and assistance to make this year more successful than the last.

The University of Florida and the Prestressed Concrete Institute are co-sponsoring the Second National Short Course on Prestressed Concrete covering design and production. It will be held at Daytona Beach, Florida, January 27-31, 1958. Engineers, architects, producers and those in the building industry in general would benefit greatly by attending. The program includes lectures on design and advanced design as well as talks on production problems.

The PCI members and other interested parties will receive all the details by mail around the 1st of October.

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Moment Redistribution Beyond Elastic Limit and at Failure In Prestressed Concrete Beams

BY GIORGIO MACCHI

(Paper presented to the "Symposium on Plasticity in Strength of Materials and Structural Design"—Varenna, 1956, September. Translation from: "Nuovo Cimento" — Supplemento 1957.)

INTRODUCTION

A paper presented last year at the Congress of the "Federation Internationale de la Precontrainte" (1) described the tests which had been carried out in Turin on three prestressed beams continuous over three spans and on a simply supported control beam (Fig. 1).

Tested to failure by applying two equal point loads very near to the midspan, the continuous beams showed a considerable inelastic behaviour revealed by a favourable redistribution of moments with regard to the diagram of the elastic theory.

However, failure occurred at midspan when development of full strength was not yet complete. This is evident if we look at the photographs of the midspan and of the supports, Figs. 1(a), 1(b) and 1(c).

It was possible to draw the bending moment diagrams measuring experimentally the ultimate bending moment at the midspan. (Fig. 2.) From the diagrams it appears that we were still very far from the complete redistribution of moments which is postulated by the plastic-hinge theory. If we consider the ratio P/Ppl between the effective failure load P and the full redistribution load Ppl corresponding to a complete adaptation (ratio which we will term "efficiency" of the system), we see that its values were—in the case of our beams—only r = 0.87 — 0.87 — 0.78.

At the same Congress similar results were reported by Professor Lin, University of California (2) (efficiency r = 0.87 — 0.91), and were obtained on two beams over two spans.

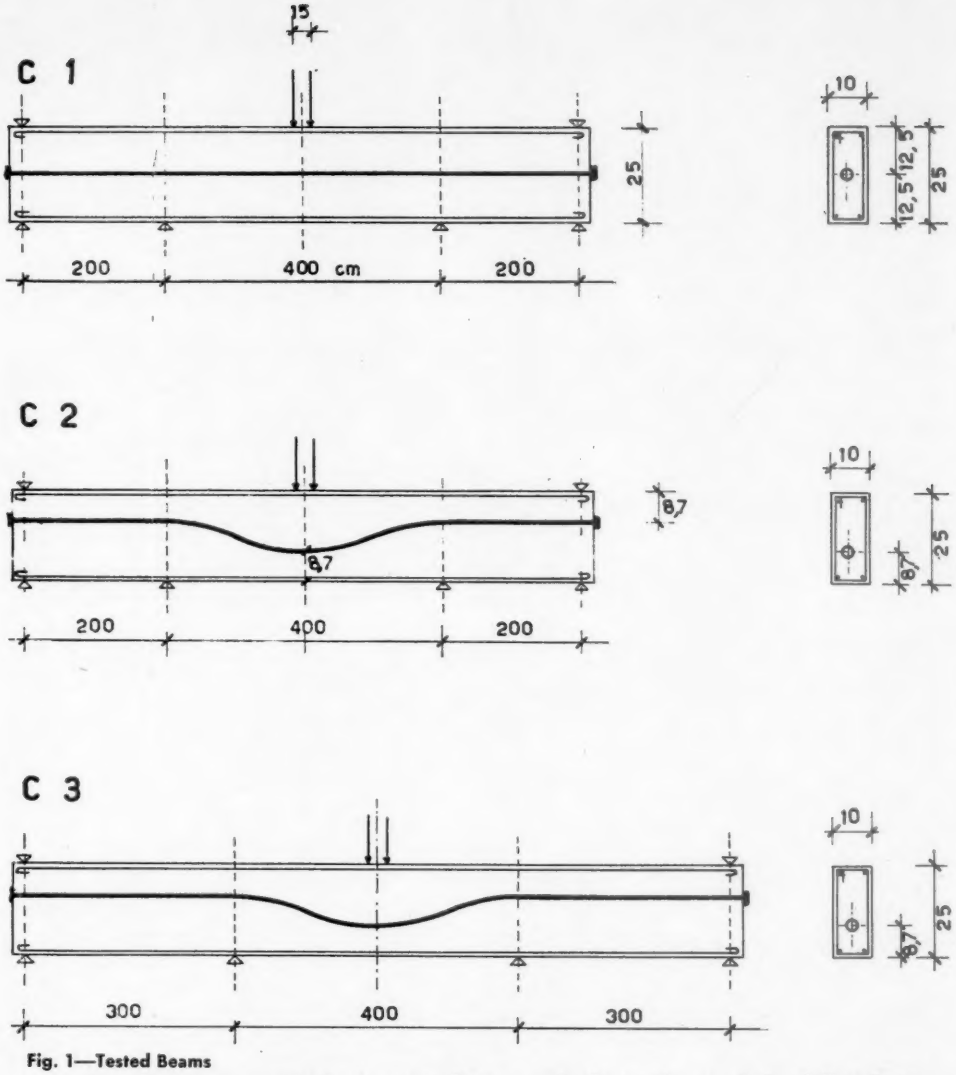


Fig. 1—Tested Beams

Yet they were seemingly in contradiction with the evidence of the full redistribution made by Guyon on four beams over two spans which were tested in Rouen(3), and by Morice and Lewis on 28 beams over two spans which were tested at the "Cement and Concrete Association". (4).

In his "General Report" (5) Guyon considered the results of Professor Lin and ours as a limitation to the validity of the plastic hinge theory that could be expected (°). In May of this year, at the Symposium held in London, he justified theoretically the low value of the efficiency ratio observed for our beam C_3, by means of an analysis of the compatibility at failure based on an idealized moment-curvature diagram. The same method applied to the British beams checked, he says, the observed full redistribution.

Yet, the difference between the two series of tests remained virtually unexplained while it seemed, on the other hand, that there could be no doubt on the experimental results. In fact, although the numerical values of the efficiencies and bending moments may be argued, the evidence of failure at the loaded section only in one case and the visual evidence of simultaneous rupture in the other case are convincing by themselves.

(°) p. 13: "On justifierait ainsi les limitations trouvees par M. Macchi et il ne s'agirait pas (d'ailleurs le soin avec lequel ont ete faits les essais excluait cette hypothese), de resultats accidentels, mais d'une limitation reelle et sensible."

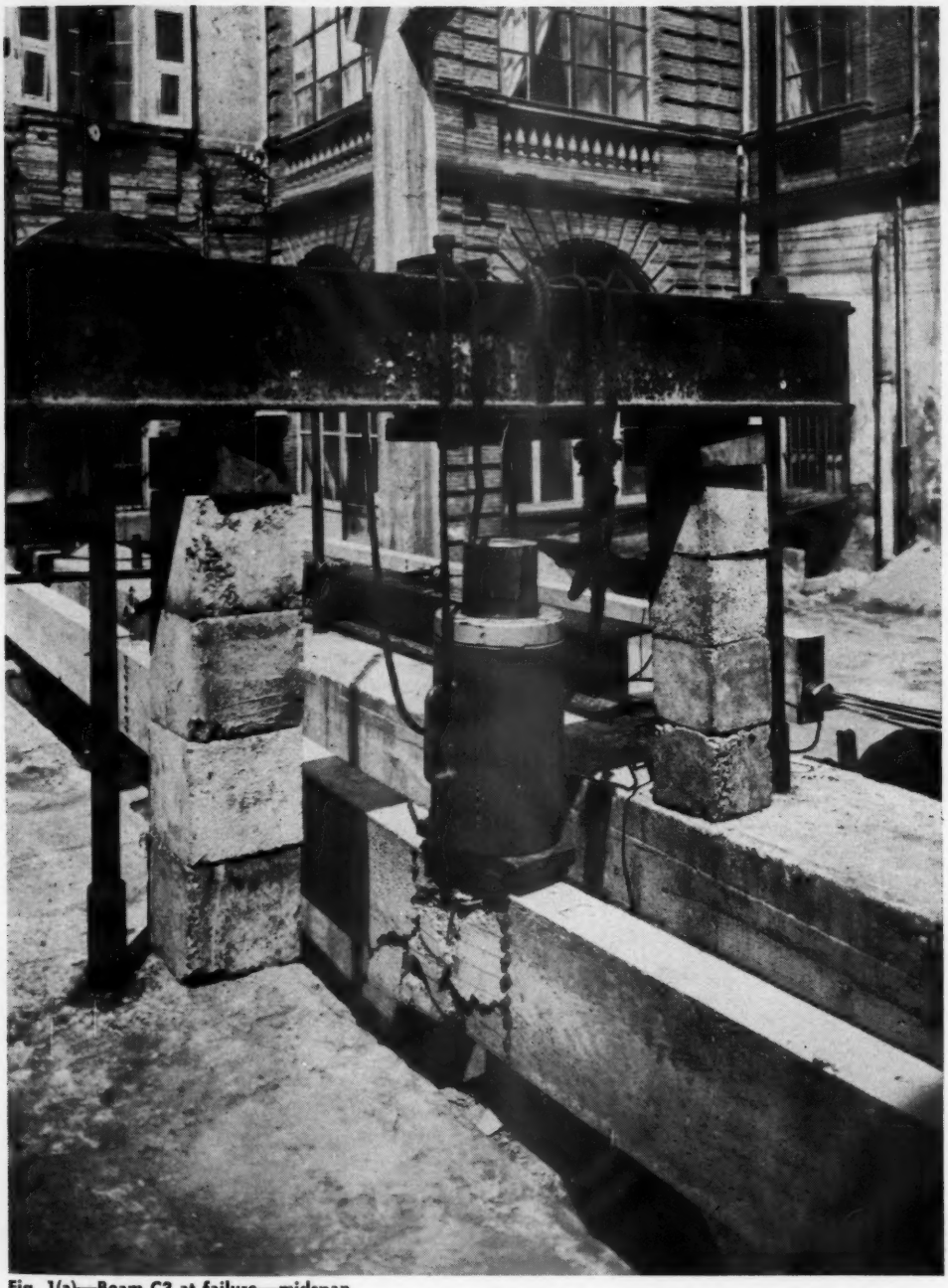


Fig. 1(a)—Beam C3 at failure—midspan

In this paper we will illustrate a method, by means of the usual theory of failure of prestressed sections of the theory of coactions and using the measurements of rotations which were taken during the tests. Results seem to justify for our beams the lack of a full redistribution and at the same time to point out the radical difference between our beams and those tested by Guyon and Morice.

Validity of the theory of coactions in the plastic field; agreement with experimental results.—

According to the theory of coactions (**) the state of equilibrium beyond elastic limit may be considered as the superposition of the elastic effect of loads on the system and of the effect (also this effect is elastic and therefore superposition is admitted) of the inelastic deformations regarded as Volterra's distorsions (*) acting in the sections where such deformations have taken place.

Fig. 1(b)—Beam C3, intermediate support B after failure at midspan

(°°) coaction = mutual constriction; stresses produced in a system by deformations not compatible with external reactive elements or with inner connections; as stresses due to temperature, shrinkage (for instance, case hardening of steel or glass), residual stresses due to plastic deformation, stresses produced by prestress.

(°) distorsion—If we cut an elastic system and produce a relative displacement of the two sides of the section, the new configuration generally involves an elastic deformation of the system which is called "distorsion." If the relative displacement is rigid (i. e. without warping of the sides of the section) distorsions are called Volterra's distorsion; the simplest examples are given by a simple rigid rotation and a simple rigid translation.

This corresponds to imposing the compatibility of total (elastic and inelastic) deformation. This interpretation of inelastic phenomena is due to Colonnetti, (6). Some applications of this theory to a theoretical study of redistribution of moments were made by Levi (See for instance (7)).

In the case of beams, and only with regard to the distribution of bending moments, plastic deformations can be considered as distortions characterized by a relative rotation of the sides of the sections around their centroid.

In fact, plastic deformations take place on rather extended zones at the two sides of the sections of the maximum bending moment; it is not however a serious error to consider the whole rotation localized at the section of the maximum bending moment. In fact a great deal of the rotation takes part in the immediate vicinity of that section, especially when the plastic effects reach a high value, due to the small slope of the moment-curvature diagram. Besides, this error has been calculated to be a small percentage of the plastic effect, by means of the influence lines of bending moments due to distorsions changing position along the beams.

Let us consider our tests.

Having made the above assumption for plastic rotations measured on the beams during the load tests beyond the elastic limit, we have calculated the consequent coaction state consisting of a linear distribution of bending moments. Superposing these effects on the elastic moments corresponding to loads, we have calculated the new moments and have compared them with the effective bending moments measured on the structure (Fig. 3).

Fig. 1(c)—Beam C3, intermediate support C after failure at midspan

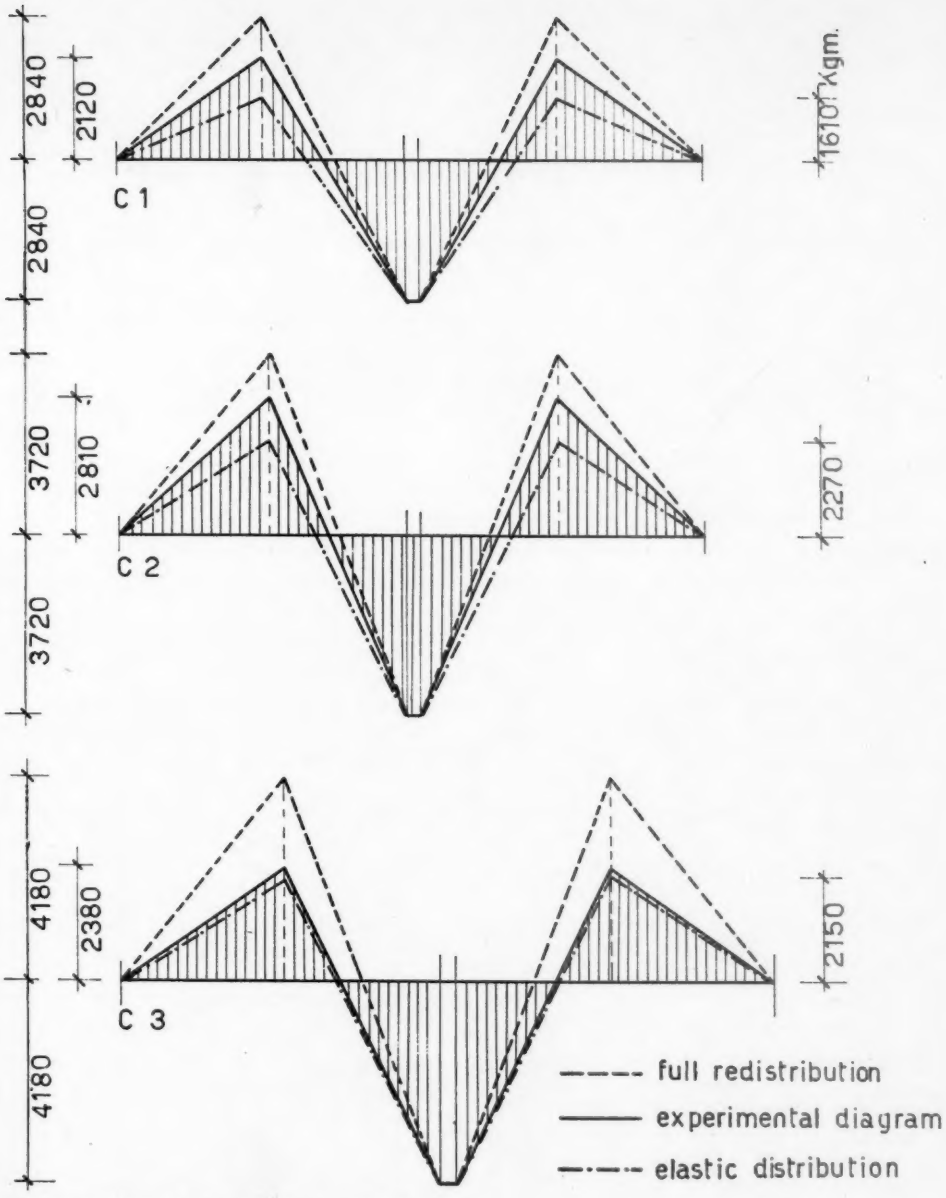


Fig. 2—Experimental and calculated moment diagrams

The slight discrepancy is due without doubt to the experimental difficulty of measuring both the moments and the rotations. Altogether it seems that the idealization of the phenomenon with the theory of coactions can be considered sufficiently near the truth. Someone could say that our comparison between theory and experience is limited to a load which does not exceed 0.75 of the failure load, while it is well known that plastic phenomena are much more considerable when the load approaches the failure load. We intend in further work to check these results in the very last stages. But we think that already it is possible to obtain a sufficient confirmation by means of ultimate strength calculations with the above mentioned assumptions and comparing the results with the sufficiently reliable data of the experimental moment diagram at failure.

Theory applied to a plastic method of calculation.—

In order to develop a practical method of calculation according to the above theory, it is necessary to make a basic assumption: the unicity of failure configuration, i.e. having found a failure configuration for which the equilibrium and compatibility conditions are fulfilled we suppose that this is certainly the real failure configuration (5). This assumption, which is perhaps uncertain for complex structures, seems to be certainly verified for the simple structures we have considered here; this method enables in any case to follow the plastic redistribution in its successive developments and therefore to study eventual exceptional cases.

Having made these assumptions, we write the two conditions of equilibrium and compatibility; this can be very easily done for the simple structures which we consider.

Let us consider for example a three span beam with a midspan load (the method is exactly the same for 2-span beams and beams with built-in ends, for any loading conditions).

The equilibrium condition is obviously obtained by assuming a linear moment diagram; the sum of the absolute values of moments at midspan and at the supports being:

$$-M_B + M_E = P\,l/4$$

The compatibility condition is easily written with the equation of 3 moments.

We calculate the fictitious reactions $\frac{B^*_{n-1}}{E\ J}$ and $\frac{A^*_n}{EJ}$ of the elastic curvatures considered as fictitious loads on the single spans, and the fictitious reactions β_{n-1} and α_n of the plastic curvatures; compatibility on a generical support gives the extended equation of three moments:

$$\frac{M_{n-1}}{E\ J}\,l_{n-1} + \frac{2M_n(l_{n-1}+l_n)}{E\ J} + \frac{M_n + l_{1n}}{E\ J} = -6\left(\frac{B^*_{n-1}}{E\ J} + \frac{A^*_n}{EJ} + \beta_{n-1} + \alpha_n\right)$$

The study of the phenomenon by means of the theory of coaction leads nevertheless to more interesting results as it enables to evaluate the effect of each plastic rotation separating it from the elastic effect of the loads.

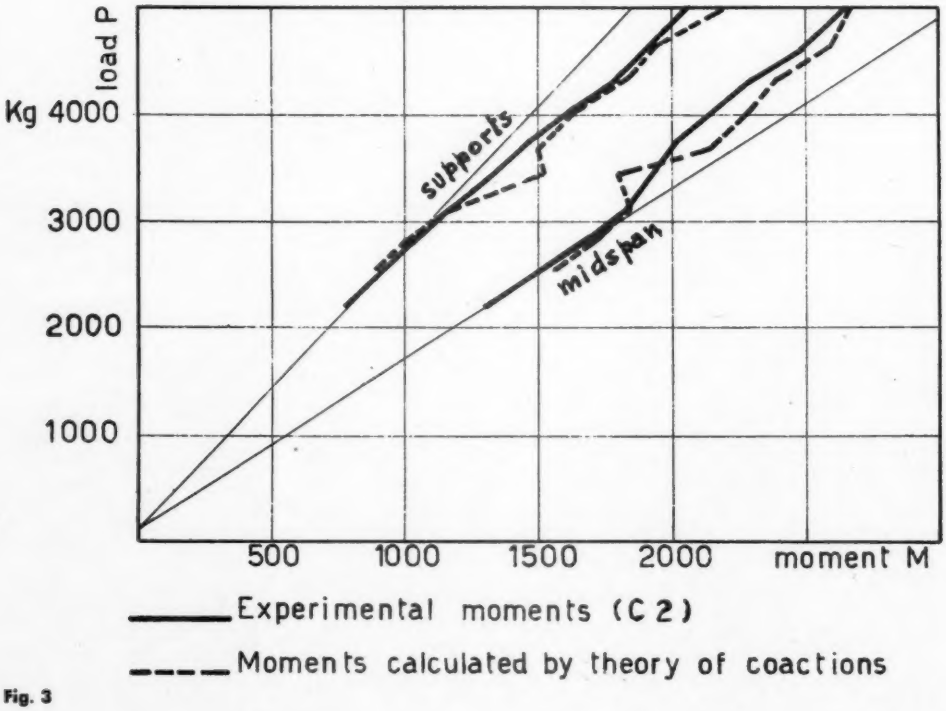


Fig. 3

On a symmetrical 3-span beam (having spans kl — 1 — kl; Fig. 4) a positive distortion having a value of unity taking place at midspan E produces a trapezoidal diagram of negative moments, having as maximum ordinate (calculated for instance from the extended equation of 3 moments):

$$\Delta M_1 = - \frac{3}{3 + 2k} \frac{E J}{l} .$$

A negative distortion having a value of unity taking place symmetrically on the two supports B and C produces an analogous trapezoidal positive diagram, having a maximum ordinate, due to the reciprocal theorem proved by Volterra:

$$\Delta M_1 = + 2 \times \frac{3 E J}{(3+2k) l}.$$

Calling α_E, α_B, α_C the absolute values of the inelastic rotations at midspan and at the two supports, their contemporatry effect is a trapezoidal diagram having maximum ordinate:

$$\Delta M = \frac{3}{3 + 2k} (-\alpha_E + \alpha_B + \alpha_C).$$

This diagram is therefore the substantial effect of redistribution of moments; superposing it to the elastic diagram due to the external loads we must obtain the equilibrium diagram mentioned earlier ($-M_B + M_E = P\ ¼$).

The difference between the present method and Guyon's method can be summarized in two points:

—Guyon's approximation is limited by the consideration of compatibility of the inelastic deformation only; on the contrary, this method takes into account also the elastic deformation along the beam;

—but a more important difference is given by the separated evalution of the effect of each distortion; it will be seen that only considering plastic phenomena from this point of view we can explain the observed different behaviour of beams of different types.

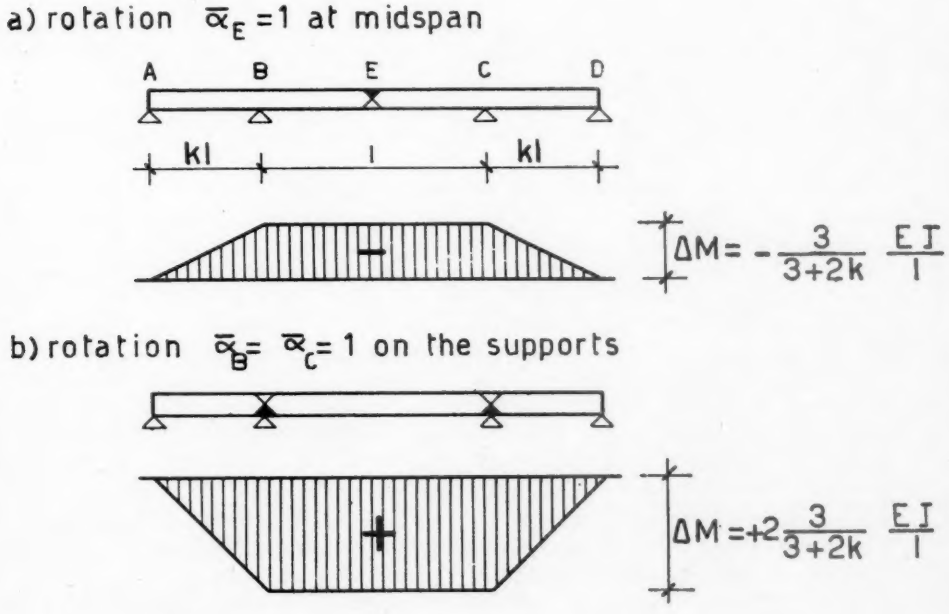


FIG. 4. Effect of distortions on a 3-span beam

Let us examine therefore how, in different structures, the effect of equal distortions can be different.

In Fig. 5 are given, for some usual cases, the positive and negative moment diagram representing the counteracting effects of unit distortions (rotations) acting at midspans and at the supports. There are also given the expressions of the maximum ordinate ΔM of the moment diagram due to the combined effect of the distortions. ΔM may be expressed in the form:

$$\Delta M = K (a \alpha_E + b \alpha_B + c \alpha_C).$$

Let us examine first the term in the parenthesis. It clearly shows that in order to have a favorable redistribution effect it is necessary that in said term the distortion in the most stressed section E prevails as compared to the other sections (if, for example, it is necessary a negative redistribution moment, the term in parenthesis must become negative). It is also clearly seen that said prevalence does not depend only on the value of each distortion, but also on the coefficients a, b, c, of α, which can have a considerable influence. In other words, it is not sufficient that considerable plastic deformations take part in the most stressed section to have a considerable redistribution effect; on the contrary, it is necessary to consider its effect in connection with that of the plastic deformations of the other maximum moment sections; and this effect highly depends on coefficients a, b, c, i.e. on the characteristics of the structure.

These considerations enable us to explain the substantial difference existing between the beams which were tested by us (3-span beams, example 1 and 2) and the beams of the French and English tests, on which a full redistribution of loads was observed (2-span beams, example 4).

In fact, in the 2-span beams with symmetrical loads in the two midspans ($a=1$ $b=0.5$ $c=0.5$) the effect of one distortion on the central support is twice the effect of an equal distortion in a midspan. This means that it is sufficient for the plastic rotation over the support to be slightly greater than that which occurs in each midspan in order to produce a favourable redistribution moment.

On the contrary, in 3-span beams ($a=1$, $b=1$, $c=1$) the effect of a distortion at midspan is equal to the effect of an equal distortion at the support. To have a prevalence of midspan and a favourable redistribution it is therefore necessary that the plastic rotation at midspan be more than twice the plastic rotation which takes place over each support. It is therefore evident, since the load adaptation is not infinite, that in this case there can be compatibility, generally, only when moments at supports are considerably lower than failure moments, i.e., with low values of the efficiency ratio.

Same considerations indicate that the beams tested by Professor Lin (example 5) were an intermediate case between the two cases we have examined. There is a possibility of high figures of the efficiency ratio for beams with built-in ends (example 6), taking into account the length and the form of the plastified zones ($\alpha_B = \alpha_C \cong \frac{1}{2} \alpha_E$, for a point load).

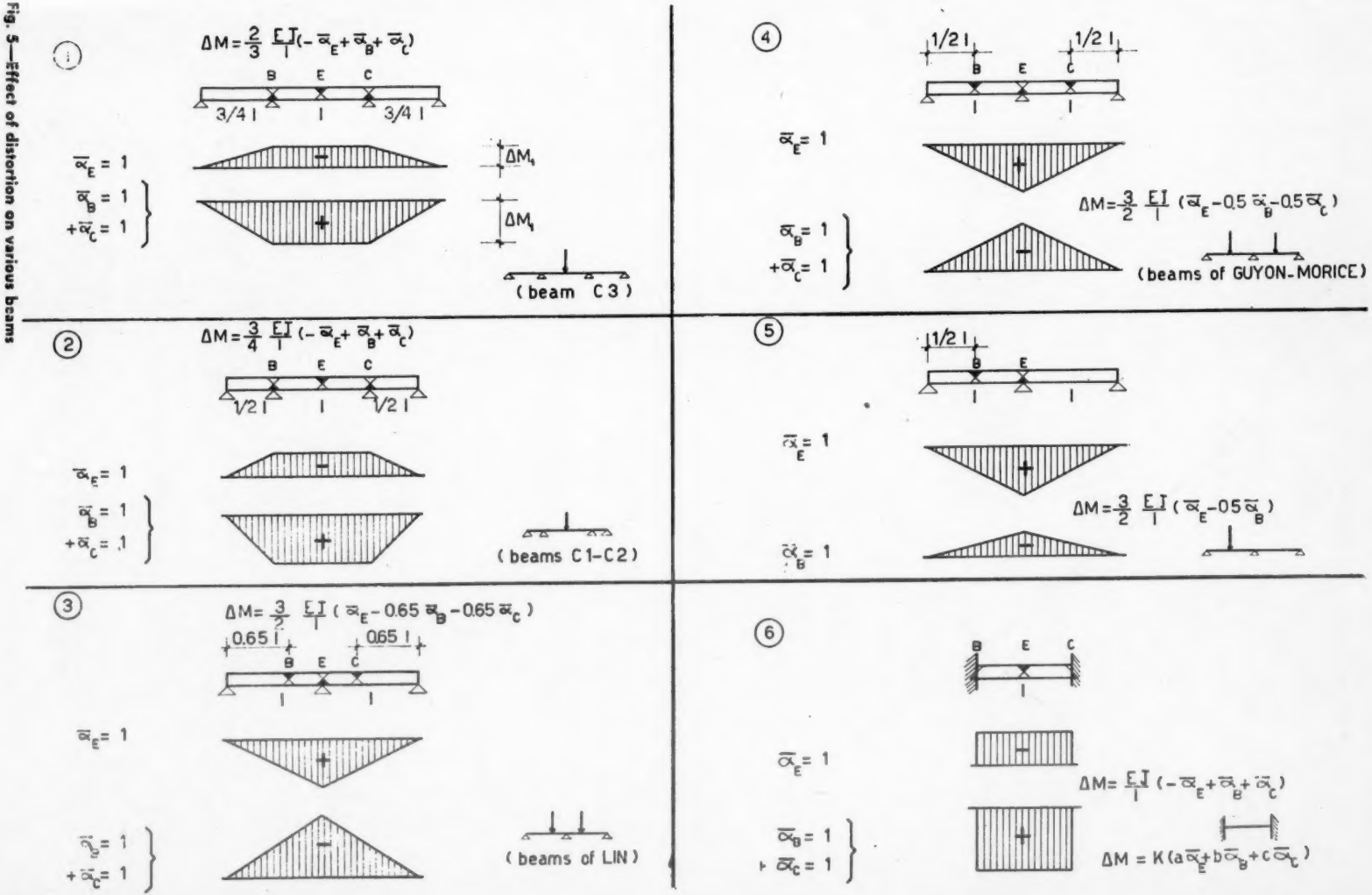
Also coefficient K has a considerable effect in the redistribution of moments. In fact, prevailing distortions being equal, redistributed moment is great for high values of K depending on the geometrical and elastic characteristics of the system. The minor value of K could for example suggest a lower figure of the efficiency ratio for beam C_3 (example 1) in comparison to beams C_1 and C_2 (example 2), as tests have shown.

Moment-Curvature Diagram

To illustrate the theory as a practical method for the plastic design of beams, it is necessary to know the relationship between bending moment and curvature in order to calculate, by integration, the plastic rotations α.

The experimental determination of the moment-curvature diagram is very difficult. Notwithstanding, Guyon was able to determine it on 6 simply supported beams, up to loads very near to the failure load (8). In our tests we could not take

Fig. 5—Effect of distortion on various beams



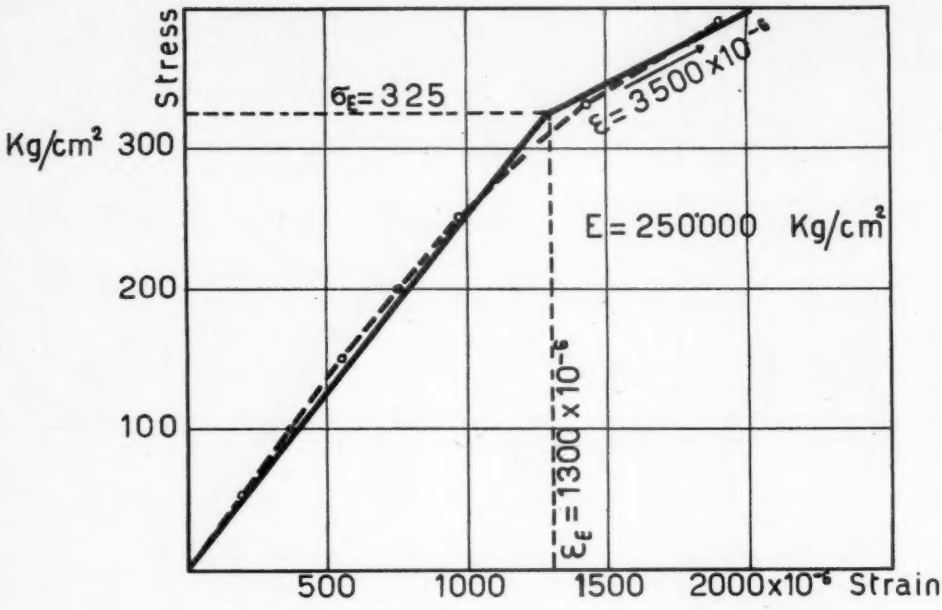


FIG. 6. Experimental and idealized stress-strain diagram (concrete)

measurements beyond 70% of the failure moment. Our diagrams stop at very low values of the plastic curvature.

To complete the diagrams it was necessary to apply the ultimate strength theory for prestressed concrete sections. From the ultimate strength diagram of the section it is in fact possible to obtain directly the curvature at failure.

This method (9) consists in the determination of the position of the neutral axis at failure, making the assumption that plane sections remain plane and imposing (by a trial and error procedure) the equality between the resultant compression and the resultant tension. In drawing the stress diagram and in calculating the resultants it is taken into account the real stress-strain diagram of the high tensile steel and a concrete stress-strain diagram idealized with two straight lines sufficiently approaching the experimental diagram (Fig. 6).

Let us examine for instance the ultimate strength diagram of section C_2. Having fixed point C, which corresponds to the strain in concrete at failure $\epsilon_{br} = 3.5\%_0$), the problem is reduced to having rotation around that point, the line CF which idealizes the assumption that cross section remains plane, to find equal values for the resultants. The compression resultant is given by the shaded area CC'EN (C' corresponds to the strength of concrete, 550 kg/cm² in our case; E represents the elastic limit in compression, 325 kg/cm²). Fig. 7.

The tension resultant is given by the tension in steel corresponding to the strain PA plus the tension at the yield point of mild steel reinforcement.

Having found the position of the neutral axis N the corresponding ultimate moment can be calculated, while the diagram shows the failure curvature μ_r given by:

$$\mu_r = \frac{\epsilon}{y} = \frac{OC}{ON}; \text{ in our case:}$$

$$\mu_r = \frac{3.5}{1000 \times 78} \text{mm}^{-1} = 44.6 \times 10^{-6} \text{ mm}^{-1}.$$

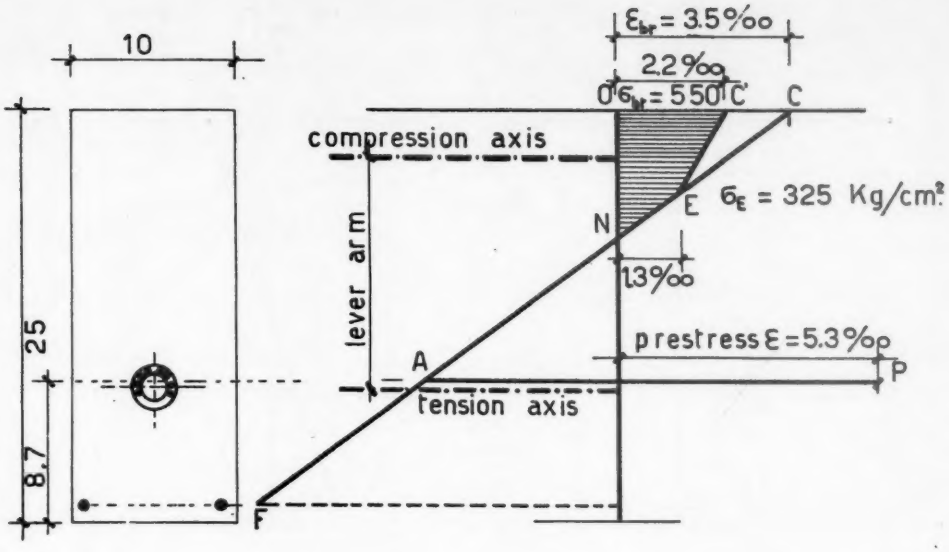


FIG. 7. Strain and stress distribution at failure. (C2)

This rotation and the failure moment (which in our case is known also experimentally) give us abscissa and ordinate of the final point of the moment-curvature diagram.

We observe incidentally that the value of curvature μ_r is not considerably affected by the assumptions on the stress-strain diagram of concrete; on the contrary, the value of the strain in concrete at failure ϵ_{br} has a significant influence; assuming $4.5\%_0$, instead of $3.5\%_0$ μ_r would increase by about 20%.

Further intermediate points of the moment-curvature diagram can be obtained in a similar way, as proposed by Guyon. In our case moment and curvature were calculated corresponding to the elastic limit of the concrete in compression and in a further intermediate point assuming the same two-straight-lines diagram (Fig. 8). The values obtained are given on the moment-curvature diagram of the section (Fig. 9). R corresponds to failure, E_2 corresponds to the elastic limit of concrete in compression (the agreement with the experimental results is satisfying), I is the intermediate point.

It should be noted that the diagram has an approximate three-line shape, which may be surprising compared with the shapes usually assumed for the moment-curvature relationship.

It seems that a final line with a still considerable slope should take the place of the nearly horizontal line as it is often represented. The capacity of the section near failure, for large curvatures, would be nearly constant.

A further very interesting result is obtained by superposing the diagrams (M,μ) of the two different sections C_1 and C_2 (the former with the axially positioned cable, the latter with the cable at the bottom middle-third), using values obtained at failure, i.e. plotting the diagram $(M/M_r, \mu/\mu_r)$. In fact, a considerable coincidence of the two sections diagrams is obtained, at least with regard to the plastic range, and therefore with regard to the values involving our calculations. (Fig. 10).

If the diagram $(M/M_r, \mu/\mu_r)$ should be constant within sufficiently wide limits of variation of the sections, it would easily overcome the greatest obstacle in plastic calculation. (It can be foreseen, for T sections for example, that the diagram in the plastic state coincides with the diagram for rectangular sections — the cracked

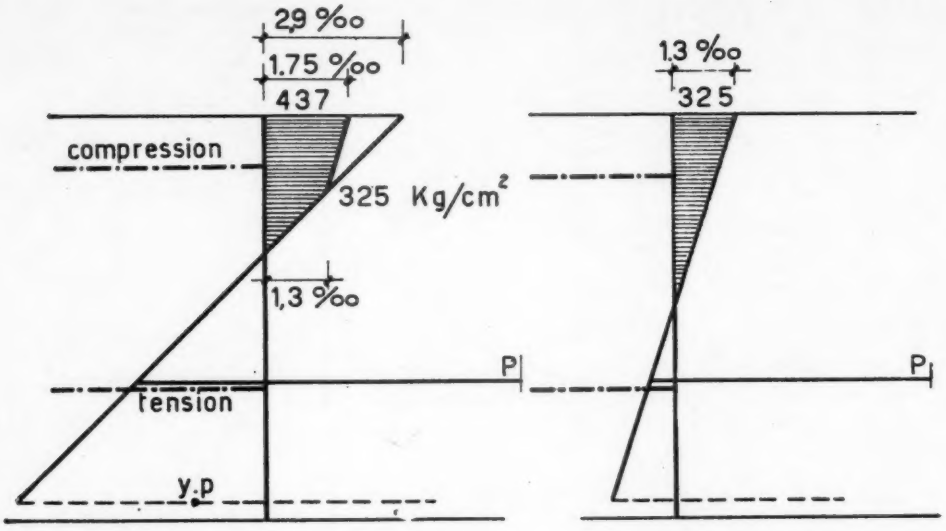


FIG.8. Strain and stress distribution at intermediate point and at elastic limit (C 2)

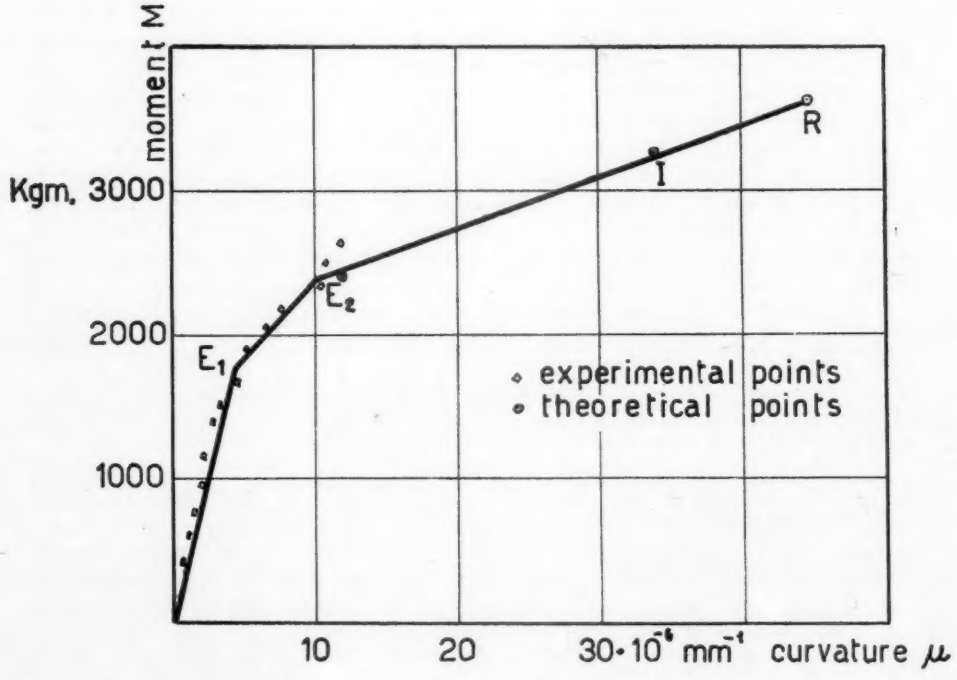


FIG.9. Moment-curvature diagram (C 2)

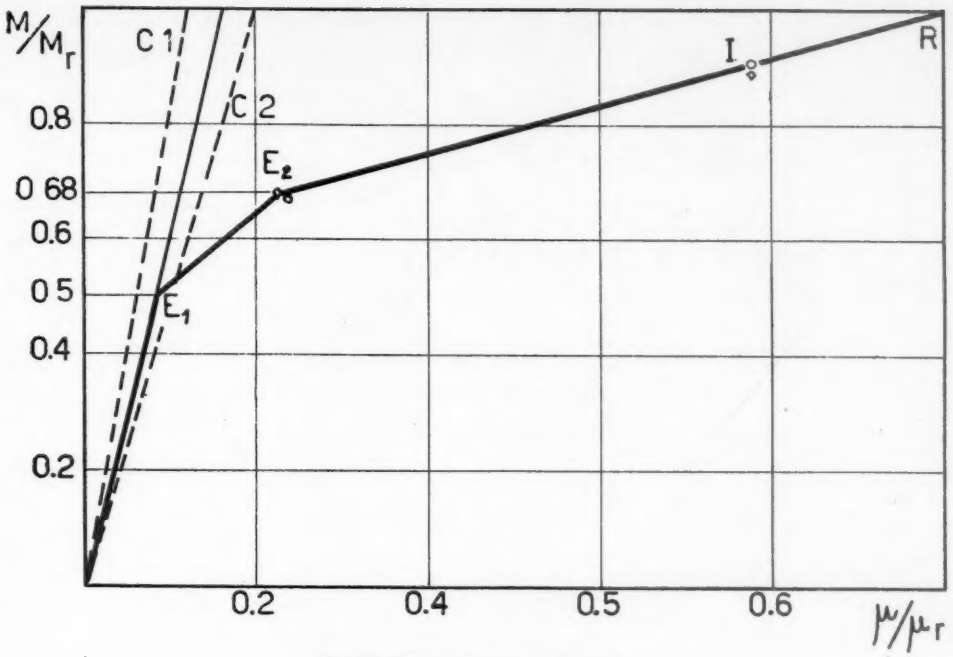


Fig. 10—Moment-curvature diagram

part of the section having no influence either on the moment or the curvature.)

As a consequence of our assumptions, it seems that the diagram M/M_r, μ/μ_r) may be idealized in the following way:

—a first straight line of elastic behaviour (representing the equation : $\mu = M/EJ$) up to the first elastic limit E_1 ($M \cong 0.5M_r$), corresponding nearly to the beginning of cracking;

—first plastic phase between the first elastic limit E_1 and the second (E_2), ($M \cong 0.68\ M_r$) corresponding to an inelastic behaviour in tension due to cracking but to a still elastic behaviour of concrete in compression;

—second plastic phase, beyond the second elastic limit E_2 corresponding to an inelastic behaviour both in tension and in compression.

This last part of the diagram has a slight slope, but it seems to reach the failure moment still considerably inclined.

The Method Applied to Plastic Calculation of Beam C_2 —

For the diagram ($M—\mu$) which we obtained based on a series of assumptions, an experimental verification is of course necessary. In particular, two of the most uncertain assumptions are:

—the assumption $\epsilon_{br} \cong 3.5\%_0$

—the assumption that curvature calculated with such value can be extended without reduction to a zone of constant moment and is not localized in the failure section (assumption favorable to redistribution).

However, to have an idea of the approximations that are possible to obtain with these assumptions we will calculate the ultimate strength of our beam C_2 and compare them with the experimental data.

It is easily checked that the moment diagram corresponding to a complete redistribution leads to a configuration for which compatibility is not fulfilled.

On the contrary, it is possible to check that compatibility of deformation is fulfilled only assuming at the supports a moment $M_B = M_C = 3120$ Kgm. i.e.,

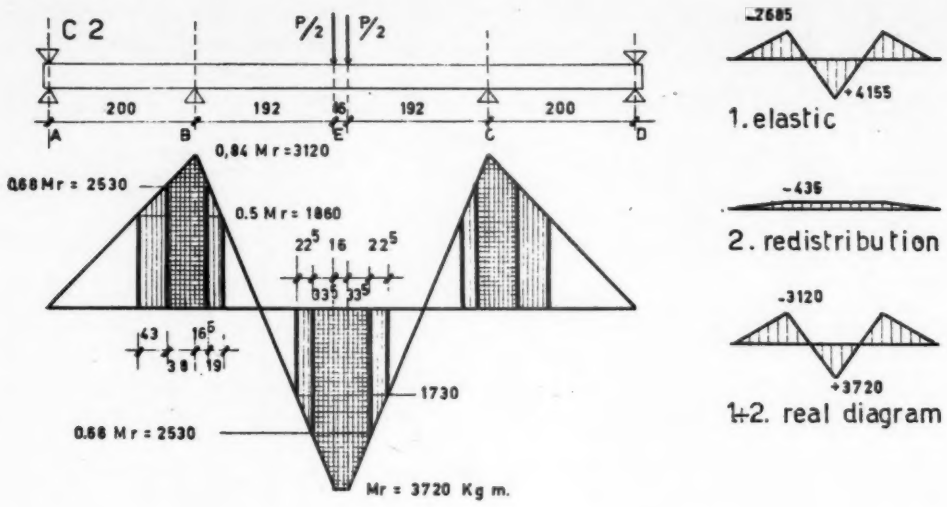


FIG. 11. Moment diagram at failure (C2)

0.84 M_r, being the failure moment $M_r = 3720$ Kgm. (Fig. 11).

Considering the moment diagram of simply supported beam between the two intermediate supports and taking into account the dead load q of the beam, the load P which would give rise to the assumed diagram is such that:

$$\frac{P}{2} 1.92 + q \frac{1^2}{8} = 3120 + 3720$$

hence:

$$P = 7000 \text{ kg.}$$

According to the elastic theory, this load P and the dead load would give rise to the moments:

$M_E = 0.586 \times 7000 + 55 = +4155$ Kgm

$M_B = M_C = -0.374 \times 7000 - 68 = -2685$ Kgm

To obtain the assumed moment diagram at failure it is necessary to superpose the elastic diagram now found and a trapezoidal diagram having maximum ordinate:

$$\Delta M = -3120 + 2685 = 3720 - 4155 = -435 \text{ Kgm.}$$

It is sufficient to check that the combined effect of the three distortions at midspan and on the two supports, calculated with the theory of coactions, gives rise to this trapezoidal diagram.

First of all, the extent of the zones in the first and second plastic phase is calculated (shaded zones) delimited by the ordinates $0.5M_r$ and $0.68M_r$ according to our diagram ($M;\mu$).

(The figure 1730 takes into account the depth variations of the cable; incidentally, it is also noted that the zones so calculated coincide appreciably with the zones showing cracking during the tests.)

The moment diagram being nearly linear, and the diagram M/μ assumed to be linear in each plastic phase, the calculation of total plastic rotations α_E, α_B, α_C may be simply done by introducing the average values μ_m of plastic curvatures:

midspan:

First plastic phase:

$$\mu_m = 2.05 \times 10^{-6} \text{ mm}^{-1}; \quad \alpha_{E1} = \Delta x_1 \times \mu_m = (225+225)\ 2.05 \times 10^{-6} = 9.2 \times 10^{-4} \text{ rad.}$$

Second plastic phase ($M_{max} = M_r$)

$$\mu_m = 19.8 \times 10^{-6} \text{ mm}^{-1}; \quad \alpha_{E2} = \Delta x_2 \, \mu_m = (335+335)\ 19.8 \times 10^{-6} = 132.6 \times 10^{-4} \text{ rad.}$$

$$\mu_r = 35.7 \times 10^{-6} \text{ mm}^{-1}; \quad \alpha_{E2'} = \Delta x_2 \, \mu_r = 160 \times 35.7 \times 10^{-6} = 57.2 \times 10^{-4} \text{ rad.}$$

$$\alpha_E = \alpha_{E1} + \alpha_{E2} + \alpha_{E2'} = (9.2+132.6+57.2)\ 10^{-4} \text{ rad.} = 199 \times 10^{-4} \text{ rad.}$$

Supports

First plastic phase.

$$\mu_m = 2.01 \times 10^{-6} \text{ mm}^{-1}; \quad \alpha_{B1} = \Delta x_1 \, \mu_m = (430+190)\ 2.01 \times 10^{-6} = 12.5 \times 10^{-4} \text{ rad.}$$

Second plastic phase ($M_{max} = 0.84 M_r$)

$$\mu_m = 0.27 \, \mu_r = 0.27 \times 44.6 \times 10^{-6} = 12.1 \times 10^{-6} \text{ mm}^{-1}$$

$$\alpha_{B2} \, \Delta x_2 \mu m = (380+165)\ 12.1 \times 10^{-6} = 66.0 \times 10^{-4} \text{ rad.}$$

$$\alpha_B = \alpha_C = \alpha_{B1} + \alpha_{B2} = (12.5+66.0)\ 10^{-4} = 78.5 \times 10^{-4} \text{ rad.}$$

Therefore in the term $(-\alpha_E + \alpha_B + \alpha_C)$ midspan prevails:

$$-\alpha_E + \alpha_B + \alpha_C = (-199+2 \times 78.5)\ 10^{-4} = -42 \times 10^{-4} \text{ rad.}$$

The effect of the three contemporary distortions is therefore (Fig. 5):

$$\Delta M = \frac{3}{4} \frac{EJ}{l}(-\alpha_E + \alpha_B + \alpha_C) = -\frac{3}{4} \frac{400{,}000 \times 13{,}020}{400} 42 \times 10^{-4} = -41{,}000 \text{ Kgcm} = -410 \text{ Kgm.}$$

The comparison between this value and $\Delta M = -435$ Kgm previously calculated, shows that compatibility of deformation is practically obtained.

In this case the theoretical efficiency is then:

$$r = \frac{7000}{7610} = 0.92$$

giving an error of 5% compared with the experimental value observed.

We cannot explain this difference which is of the order of magnitude of normal scatter.

It should be noted, however, that we have so far obtained, also in other examples with the present method, values of the efficiency r greater than the experimental values; we may be wrong, but we believe that this is a systematic error.

Perhaps it would be necessary to reduce the value of μ_r of the curvature at failure. Or, are we in error in supposing that this local curvature extends to the whole zone of constant moment? The latter thesis seems to be supported by the fact that the experimental value r=0.87 of the efficiency ratio can be obtained with the present method if we suppose the beam of having one-point load instead of two-point load, 16cm apart (angular diagram instead of trapezoidal diagram).

Application to Further Examples—Proposal for a Direct Method

The present method was also applied to beam C_1 with practically the same results as for beam C_2.

Lacking sufficient data on the two-span beams tested in France and in Great Britain, on which a full redistribution of moments was observed, we have applied the method to an imaginary two-span beam having section C_1 at the midspans and section C_2 on the central support with two concentrated loads being symmetrically applied at the midspans.

Also in this case calculations show that compatibility is not fulfilled with the assumption of full redistribution; compatibility was obtained with an efficiency of only r=0.94. It is however checked that two-span beams can give rise to higher efficiencies, as we have showed discussing the expression of ΔM. In fact, the value r=0.94 is to be compared with r=0.87 obtained for the three-span beams having the same shape of the moment diagram (angular shape).

Calculations carried out with the same method show that full redistribution would be allowed by one of the three following conditions:

1⁰)—a wide support zone, of at least 1/20 of the span (trapezoidal diagram)

2⁰)—curvature μ_r (at failure) over the support being about twice the curvature at the midspan.

3⁰) asymmetrical loading (ratio of about .08 between the two loads.)

The first thesis seems to partially correspond to the test conditions; the British beams had on the central support a thick steel plate 4" wide.

The second thesis seems to be unlikely: in fact, the rotation capacity increases with a diminished depth of steel, but not in a considerable way, as Morice and Lewis have already rightly noted (the ratio between the curvatures of C_1 and C_2 is only 1.15).

The third thesis should be judged only by experimenters, but a ratio of 0.8 is certainly impossible; it seems however that some asymmetry of the loads is difficult to avoid when testing with two jacks.

It could be thought that all the three conditions verified only partially and that superposition of their effects caused fulfillment of redistribution.

It is therefore clear, due to the above considerations, that the British and French beams should produce a greater redistribution than our beams.

A last interesting example to which we have applied the present calculation method is given by a three-span continuous beam having a point load at midspan but without any apparent disproportion between midspan and supports; i.e. having failure moments proportional to the elastic distribution moments. Such structure in which, at first sight, no moments redistribution could seem necessary, would reach failure only at midspan, according to our calculations, with an efficiency of $r = 0.94$. In fact, due to plastic deformations, the moment diagram at failure does not remain proportional to the elastic diagram. (*)

The above method can be applied to reinforced concrete beams in the same way that we have described for prestressed beams; only, we can obviously foresee that moment-curvature diagram shall be different due to the different characteristics of steel and to an earlier beginning of cracking.

Let us at last shortly examine the possibility of reducing the present plastic method to a direct form in order to avoid the painful series of trials.

If our assumptions should result sufficiently close to reality the establishment of direct calculation would not be difficult.

For example, in the case of the three-span beam (having spans kl — 1 — kl) subject to a concentrated load at midspan, if it is assumed as unknown the ratio $s = \frac{M_B}{M_{Br}}$ between the real moment attained at the support at failure and the ultimate moment of the section, compatibility is expressed by a simple third degree algebraic equation (due to linearity of M/μ relationship and of the moment diagram).

As an example, and with obvious reservations on the introduced numerical coefficients, for the above case the equation would be index 1 for midspan, 2 for support, $d_r = \frac{M_{2r}}{M_{1r}}$)

$$s^3 \left\{ \frac{3+4\frac{k}{6+4\frac{k}{}}}{} (M_{r1}) d^2_r - \frac{3\ E\ J}{3+2\ k} (1.11) d_r \mu_{2r} (1+2k) \right\} +$$

$$+ s_2 \left\{ -(2 \frac{3+4\frac{k}{6+4\frac{k}{}}}{} - 1) M_{r1} d_r + \frac{3\ E\ J}{3+2\ k} \mu^2_r (-2.22k + 2.76k\, d_r + 1.38\, d_r) \right\} +$$

$$+ s \left\{ -(\frac{3+4\frac{k}{6+4\frac{k}{}}}{} - 1) M_{r1} + \frac{3\ E\ J}{3+2\ k} \left[0.1069 \mu_{1r} + \mu_{2r} (-0.872k\, d_r + 2.76k - 0.436 d_r) \right] \right\}$$

$$- \frac{3\ E\ J}{3+2\ k}\ k\ 0.872 \mu_{2r} = 0.$$

(*) A similar case is illustrated in the second paper of (7).

requiring the only previous calculation of the ultimate strength moments M_r, and of the failure curvatures μ_r(*).

Conclusions

The redistribution of moments which takes place before failure due to the inelastic deformations often reaches considerable and not negligible amount.

However, the theory of plastic hinges is an approximation which, in many cases, leads to overestimate in an inadmissible way the strength capacities of the structures.

On the contrary, an investigation of these phenomena may lead (with the help of the theory of coactions) to a more precise evaluation of the failure loads without involving excessive calculation difficulties.

It seems that the only condition for the establishment of practical design methods is the experimental checking of some assumptions concerning the calculation of rotations and the improvement of the method of plotting the moment curvature diagram. Further, the uncertainty in the calculation of the ultimate strength moment has, of course, a consequence on the statically indeterminate problem, both with regard to the moments and ultimate strength curvature.

Of great importance also are the load conditions and the extent of supports; in fact these conditions giving rise to different shapes of the moment diagram can produce very different extents of the plastified zones and then different ratios between distortions in the most stressed sections.

For instance, we can foresee, under distributed loads, a general prevalence of distortions at the midspans due to the parabolic shape of the diagram; and hence, with respect to the condition of a concentrated load, better efficiencies for the three-span beams, and lower efficiencies for the two-span beams.

Furthermore, it seems necessary to correct the current opinion that plasticity is always a further safety reserve when design is carried on with the elastic assumptions. We have shown that an ultimate strength design based on an elastic distribution of moments (method currently used by many designers) can give rise to real factors of safety considerably lower than calculated factors.

Thus, it seems necessary to investigate in all cases the real configuration at failure, result of the characteristics of the system and the load distribution, and then to calculate the real factor of safety.

As the moment distribution at failure is usually different from that at working load, it seems rather difficult to take into account both conditions when designing structures. It would be then useful to take into consideration that slight redistribution of moments before cracking seemed having taken place in our tests (1) (it seems possible to assume a tensile resistance of the concrete of 80 to 100 Kg/cm^2, i.e. about 1100 to 1400 lb/sq.in.).

Therefore, such methods of design require particular regulations for statically indeterminate structures, replacing the limiting conditions based on working stresses by a factor of safety against cracking, and introducing a factor of safety against failure with regard to the loads and not to the moments.

Finally, it is to be noted that in the present paper only simple structures are considered, for which it seemed admissible to study the final state only independently of the successive developments of the inelastic phenomena. For more complex structures this simple assumption is not always verified. (10).

(*) In this form the equation supposes that second plastic phase is reached both at midspan and at the supports; this assumption is usually verified.

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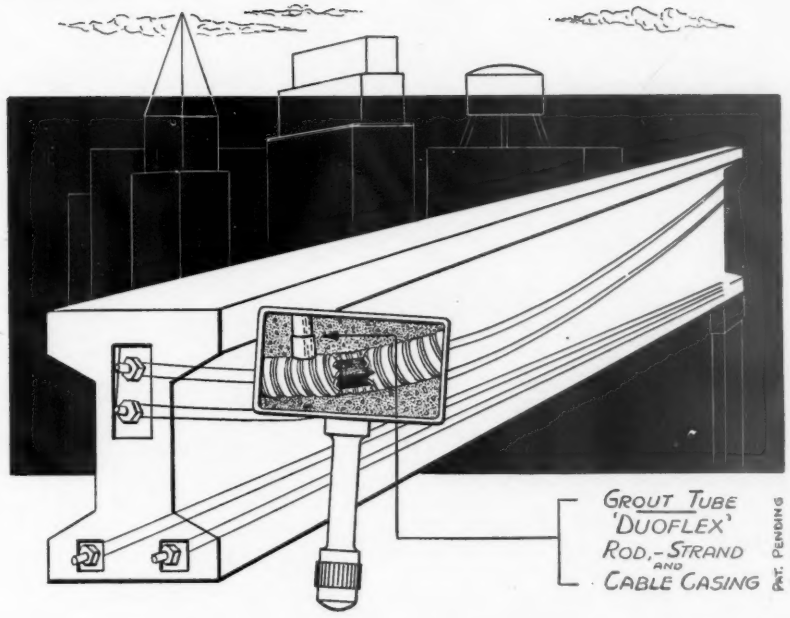

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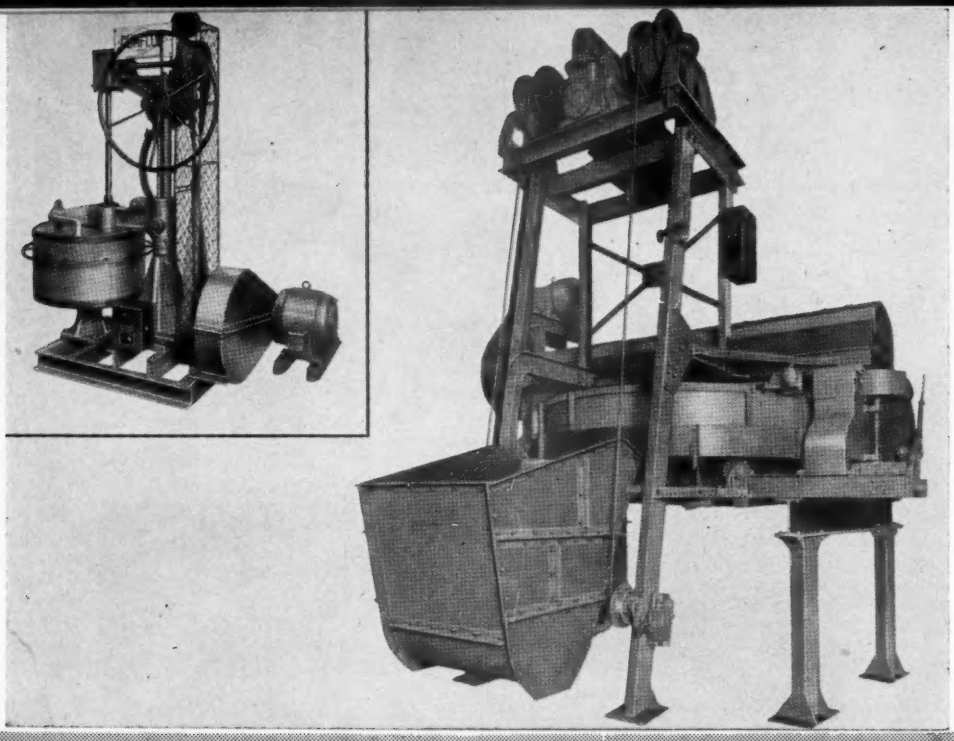

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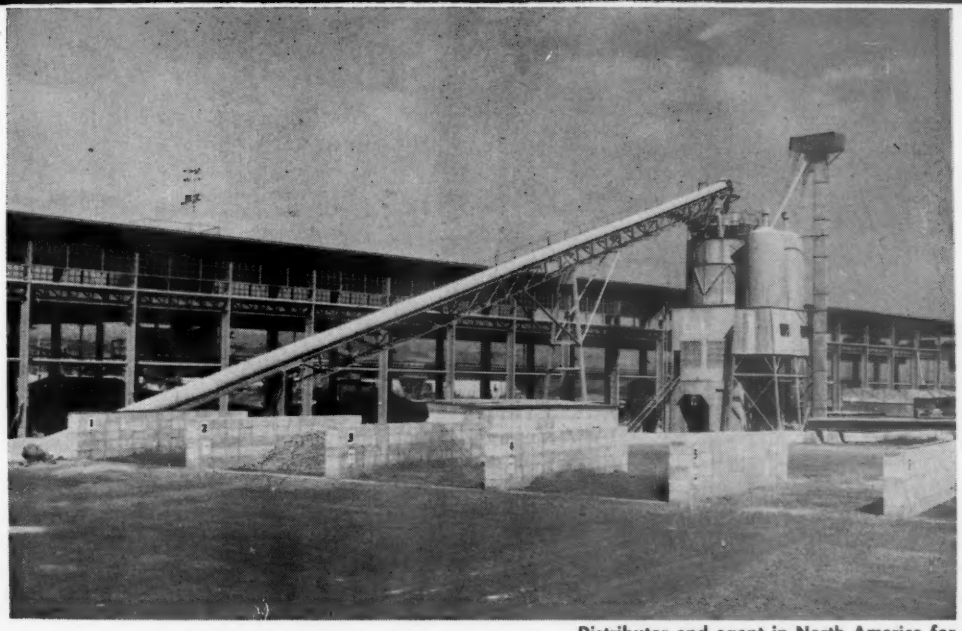

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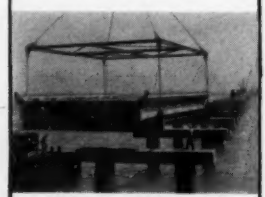

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Composite Section Used For Fast San Diego Job

By Robert C. Dorland

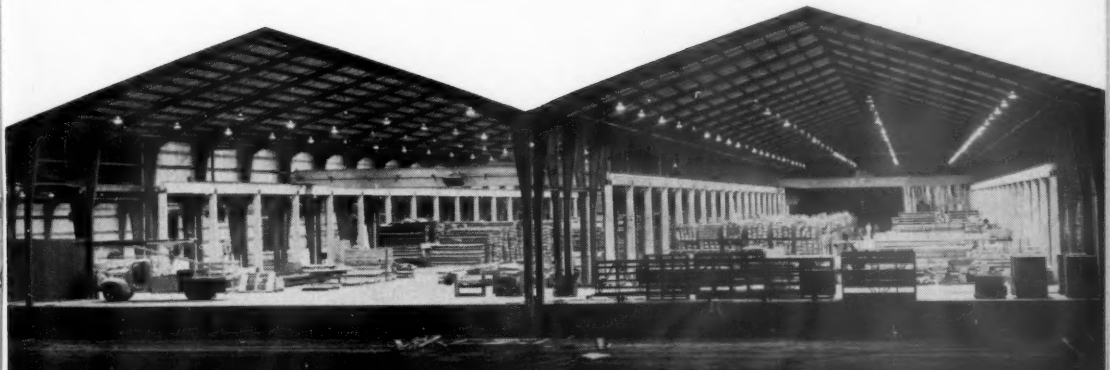
Southwest Structural Concrete Corporation

San Diego, California

■ Overhead cranes are an integral part of a new 500,000 square-foot warehouse facility for San Diego.

Crane facilities were needed on a rush schedule. Prestressed concrete was investigated as a means of providing columns and beams to support the 5 and 10-ton cranes in competition to the base design in steel or a possible alternate in precast reinforced concrete.

The successful sub-bidder quoted on the basis of precast-reinforced columns and prestressed concrete beams, with 44% of the job ready for use in 30 days and the balance in 60 days.



Prestressed concrete craneway erected in the new Convair warehouse. (Photo courtesy: Convair, a Division of General Dynamics.)

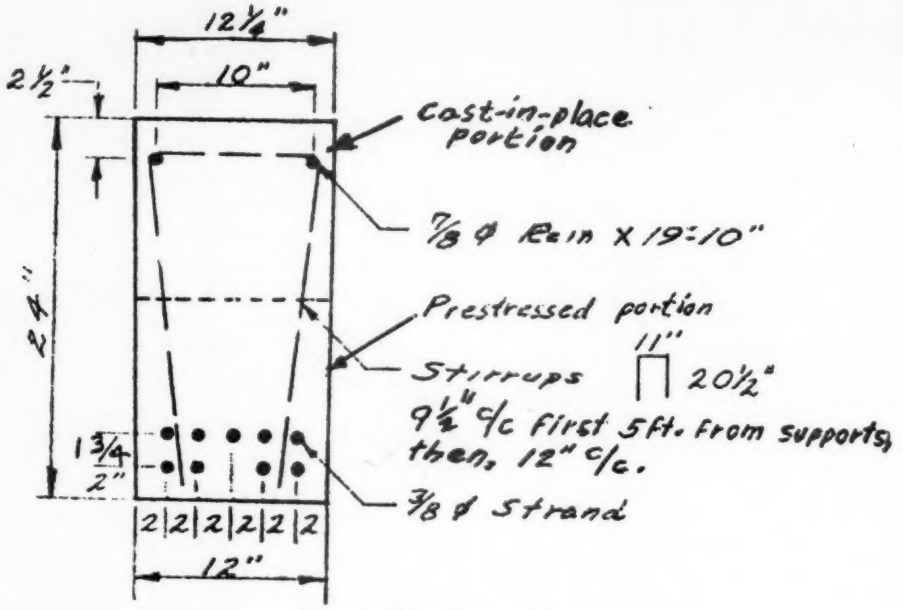


Figure 1—Cross-section of beam

Number of overhead cranes:	Two parallel, 7' separation
Length of crane travel:	804' each crane
Span of crane bridge:	72'-6-⅜"
Height of crane rail:	20'-0" above warehouse floor. 23'9" above outside storage area.
Column section:	14" thick tapered from 18" at the bottom to 12" at the top.
Column concrete:	f'c = 3000 psi @ 28 days type III cement
Prestressed beam span:	20'-0"
Prestressed beam section:	Composite 12" x 24" nominal. Lower 12" x 12" pretensioned. Upper 12" x 12" reinforced.
Beam concrete:	f'c = 4000 psi @ 28 days type III cement.
Prestressing system:	"Dorland Clip" positive end anchorage on Roebling ⅜" diam. strand.
Subcontractor on prestressing:	Southwest Structural Concrete Corp., San Diego, Cal.
General Contractor:	The James Stewart Company, Phoenix, Arizona.

PRESTRESS DESIGN

Several preliminary designs were considered for both columns and beams. Since the total linear footage of column exceeded that of beam, serious thought was given to prestressing the columns. Inasmuch as no unusual 1/r ratios or exposure problems were present and since the high seismic and impact moment at the column base could be accommodated equally well by reinforcing or prestressing, the decision was made to precast conventionally. The columns were socketed 18 inches into the footings to act as vertical cantilevers. This effected a worth-while saving in lacing between the two cranes which otherwise would have been necessary to accommodate lateral loading.

The beams are of unusual design even for the prestressing industry and are believed to be the second of their kind used for cranes in the United States*. They are of composite design, 12" by 24" deep, with a slight trapezoidal taper to facilitate removal from forms. A thin web section was not considered inasmuch as forming would have been too slow and too expensive. The lower half was pretensioned with conventional 7-wire strand equipped with positive end anchorage. The strand is of ⅜" di-

*First: Convair Wind Tunnel, San Diego, 1956.

ameter and capable of developing bond without anchorages. However, the anchorages were used to speed up the release time, vary the prestress within the beam, and reduce mild steel reinforcing requirements for end shear.

Following release of prestress in the lower half of the beam, side forms were adjusted for the top composite pour. This concrete was placed in the same way, with the same forms and on the same bed. All the required hardware to receive the rail and electrical conductor was placed in the composite pour.

Composite beams in prestressed concrete have the general advantage of manufacturing flexibility, superior stress distribution, camber control, and fewer restrictions on handling. Camber control for overhead crane is a must, and weighed heavily in the preliminary design stages. Thus the extra labor to make two pours per beam was considered worth-while. Beams were produced on two halves of a 400-ft. casting bed in the subcontractor's plant.

CONNECTIONS

The column to beam connection selected from several possibilities was a saddle made

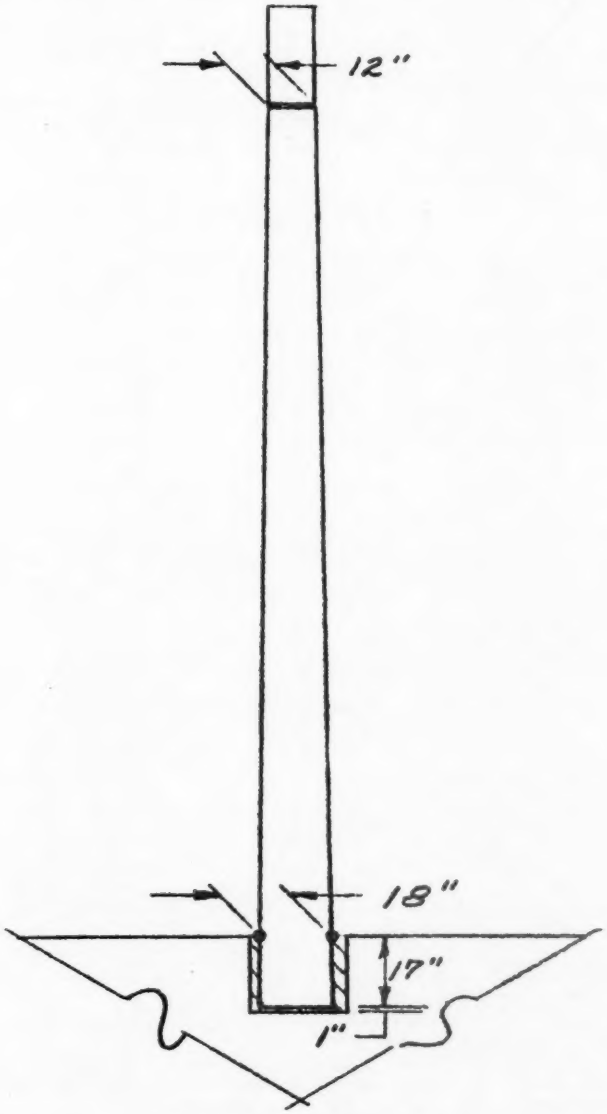


Figure 2—Column elevation

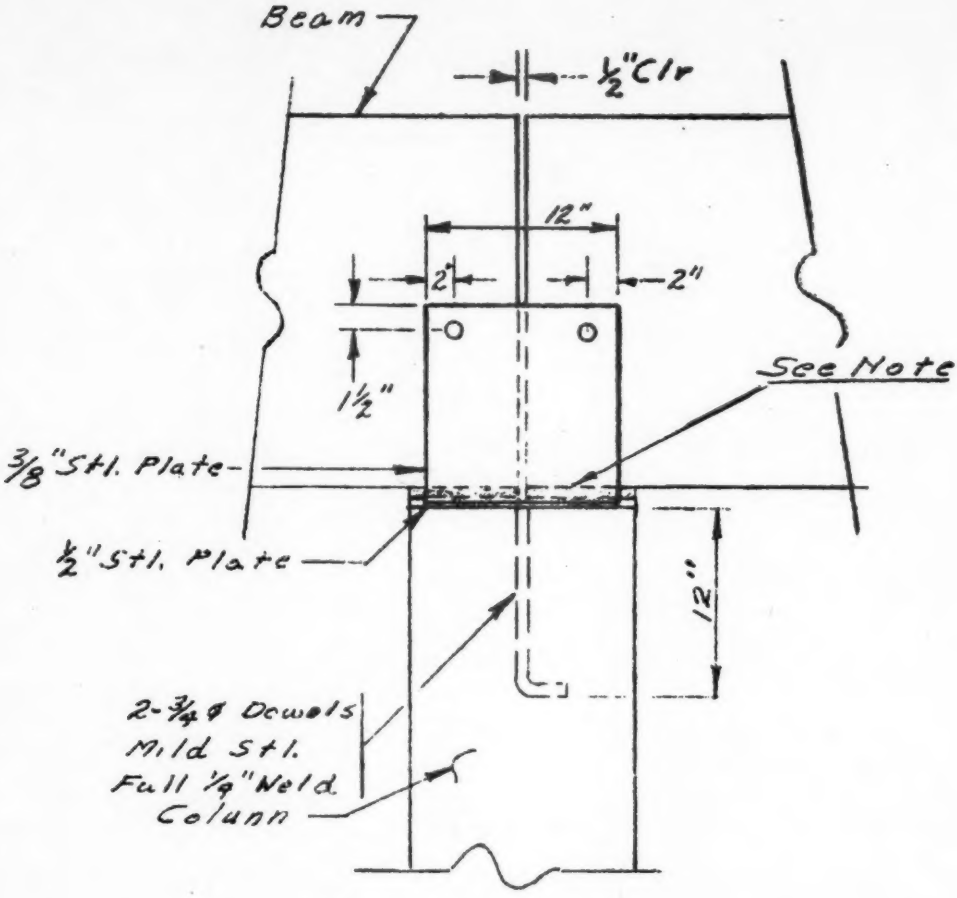


Figure 3—Beam to column connection

of welded plate and cast integrally with the column. The saddle has a sole and two side plates, each side plate is secured to the beam with a 1" bolt. The connection, somewhat expensive, proved its worth in flexibility and speed of erection. Direct bearing from the beam to steel plate saddle is taken through compressed fibre glass.

ERECTION

All members were erected from a completed floor slab inside the building with about 6 feet of headroom. One erection crew using a 5-ton hydraulic crane, supplied by two lumber carriers, maintained the required schedule and finished out on a Saturday morning a section of crane that was in use the following Monday.

EQUIPMENT	
Forms:	Plyglaze plywood (columns) Harborite plywood (beams)
Jobsite Crane:	Austin-Western Hydraulic Crane, power outrigger, front and rear steering, 5-ton cap.
Haul to Jobsite:	Two Hyster lumber carriers.

SUBCONTRACTOR PERSONNEL

Robert C. Dorland is manager and chief engineer for Southwest Structural Concrete Corp. on design, manufacture, and erection of precast and prestressed concrete.

Harry E. Dalton is general superintendent.

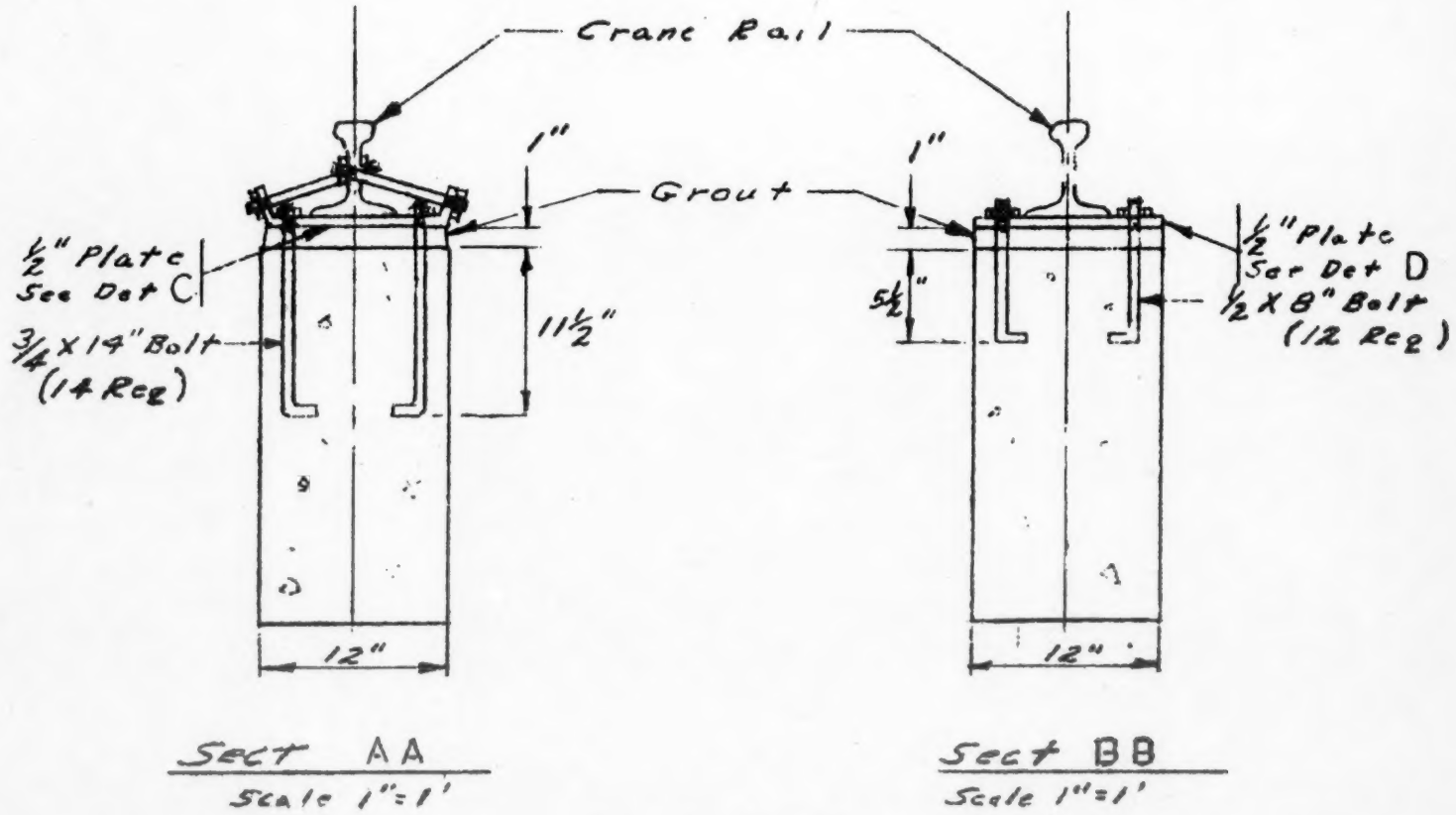


Figure 4—Crane rail connection

Figure 5—Interior view of warehouse showing craneway in place

Figure 6—View of craneway and warehouse

STEAM CURING

by
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Uniform high-early strength concrete can be obtained by steam curing. Every phase of steam curing must be properly controlled. With standard moist-cured concrete, quality will vary with changes in mixing, curing and storage conditions. With steam curing, the curing cycle is accelerated and concrete quality is much more sensitive to such changes.

A. FACTORS INFLUENCING STEAM-CURED CONCRETE

Eight basic factors will givern results obtained when steam curing:

1. Water cement ratio
2. Concrete mix temperature
3. Time delay before steam curing
4. Rate of temperature rise of kiln
5. Steam kiln temperature
6. Uniformity of temperature throughout kiln
7. Curing time in kiln
8. Relative humidity in kiln

1. Water Cement Ratio

The effect of the water cement ratio on strength of moist-cured concrete is well established. This effect will hold true with steam-cured concrete.

2. Concrete Mix Temperature

High concrete mix temperatures will result in lower ultimate strengths [1,2] (note) with any concrete, whether or not steam curing is employed. The optimum mix temperature seems to be 60°F. The maximum allowable mix temperature should be 90°F.

3. Delay Before Curing

A delay of 2 to 6 hours prior to steam curing is a must[3]. Concrete strengths can be reduced between 15 and 40% if no delay period is allowed before application of steam. The duration of this delay period will depend on the setting time of the concrete, rate of temperature rise of the kiln, and the steam kiln temperature to be employed. In general, the delay period should correspond with the initial set of the concrete. Lower mix temperatures and slower setting cements will require longer delay periods. Higher kiln temperatures will also require longer delay periods. The optimum time delay for type II cement or type I cement and Plastiment is indicated in figure 1. This delay period is often referred to as the "hold" or "preset" period.

4. Rate of Temperature Rise of Kiln

The rate of temperature rise of the concrete should be as slow as practicable, otherwise crazing and cracking will result[4]. With the proper delay before steam curing, the temperature rise of the kiln should be a maximum of 1°F per minute. Without a delay period a much slower temperature rise should be employed.

5. Steam Kiln Temperature

High steam kiln temperatures will result in high 24-hour strengths, but there will be little strength gain thereafter. Steam curing temperatures between 135° and 165°F will give 28-day strengths almost as high as moist-cured concrete, but will have lower 24-hour strength than concrete cured at temperatures above 165°F. The 28-day strength of concrete cured above 165°F will be lower than that of concrete cured below 165°F. If the steam kiln temperature is 30°F over or under the specified temperature strength at 24 hours can vary as much as 20 to 30%. Figure 2 illustrates the 28-day strengths of concretes steam cured at various temperatures. The concrete in this test produced low strength, but the same relationship would exist whether high strength or low strength concrete was used.

6. Uniformity of Temperature Throughout Kiln

It can be seen from sections 3, 4 and 5 above, that the temperature at which concrete is cured greatly affects ultimate strength. If the temperature throughout the kiln is not uniform, ultimate strengths will be non-uniform and the average low, because the time delay before curing and the length of the curing time are optimum at only one curing temperature. A practical limit on the variation of temperatures throughout the kiln would be ± 5°F.

If steam jets are permitted to impinge on the concrete member or steel forms, localized hot spots will develop with resultant accelerated strength gain in one limited section and attendant stresses and cracking.

7. Curing Time in Kiln

The length of time the concrete is subject to steam curing will greatly affect the strength of concrete at 24 hours. In general, the longer the steam curing period, the stronger the concrete will be at 24 hours[5]. Experience has shown that when steam curing at temperatures over 140°F very little is gained by steam curing longer than 24 hours.

8. Relative Humidity in Kiln

The relative humidity of the kiln should be kept as high as possible. Concrete gains strength due to a chemical reaction between portland cement and water. If water

evaporates from the concrete, lower strengths and crazing and cracking will result. Water exists in the liquid phase in concrete and evaporation is possible even when the relative humidity of the kiln is over 80%[5]. This happens often, if steam curing is continued after the concrete being cured reaches equilibrium temperature. Any and all means possible should be used to keep the relative humidity of the kiln high and the evaporation of water from the concrete low.

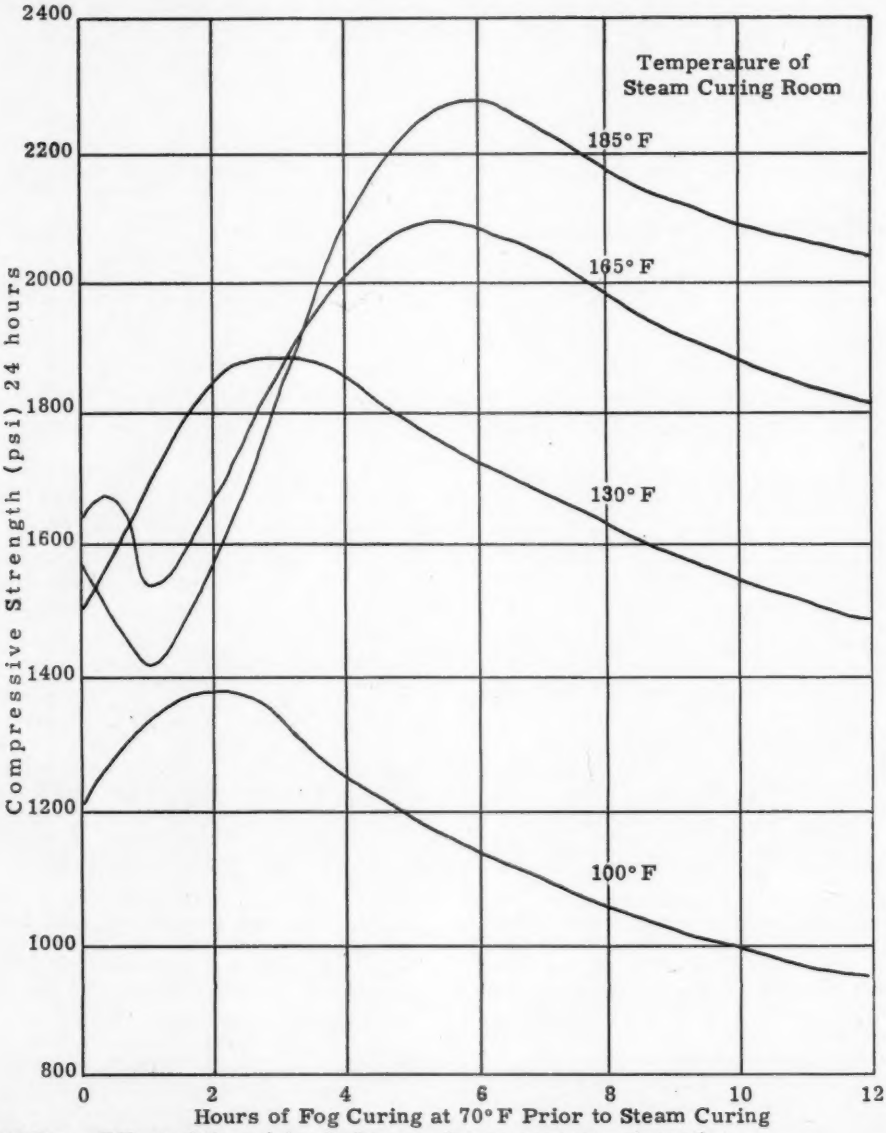


FIGURE 1 - Effect of time delay before curing on concrete strength

Higher temperatures require a longer delay prior to steam curing to provide maximum strengths at 24 hours. Curves shown are taken from the paper "Early Strength of Concrete as Affected by Steam Curing", and are representative of results with type I cement and 1 lb. Plastiment or type II cement.

Specimens were fog cured for various initial periods, then steam cured to age 24 hours and tested.

*Fig. 2—COMPRESSIVE STRENGTHS OF 6 X 12 IN. STEAM CURED CYLINDERS

Time specimens subjected to steam and fog curing	Age at Test	Compressive Strength (psi)							
		Maximum Kiln Temperature							
		100°F		130°F		165°F		185°F	
		Type I	Type II	Type I	Type II	Type I	Type II	Type I	Type II
6-hour steam cure	6 hours	330	120	280	400	620	740	880	1010
1-day steam cure	1 day	1090	1280	1400	1620	1550	1990	1420	1660
3-day steam cure	3 days	2010	2460	2620	2780	2450	2840	2000	2160
6-hour steam cure plus 7-day fog cure	7 days	2460	2770	2470	2730	2200	2210	1720	1680
8-hour steam cure plus 28-day fog cure	28 days	3510	4640	3920	4490	2820	3380	2220	2310
1-day steam cure plus 6-day fog cure	7 days	2550	2960	2700	3220	2700	2950	1670	1570
1-day steam cure plus 27 day fog cure	28 days	3760	4450	3750	4270	3200	3370	2080	2340
3-day steam cure plus 4-day fog cure	7 days	2660	2960	2970	3330	2830	2930	2320	2180
3-day steam cure plus 25-day fog cure	28 days	3460	4260	3710	4250	2800	3550	1920	2290

Fog cured control cylinders 70°F

Time Cured	Age	Compressive Strength (psi)	
		Type I	Type II
1-day	1 day	610	680
3-days	3 days	1680	1890
7-days	7 days	2640	3010
28-days	28 days	4070	4810

On all tests—delay period before curing—1 hour, maximum-temperature rise of kiln 1° F/Minute, no soaking period.

*Early Strength of Concrete as Affected by Steam Curing—J. J. Shideler and W. H. Chamberlin—A.C.I. 12/49.

CONCLUSION

The eight variables controlling the quality and strength of steam-cured concrete are additive. By ignoring even one of these points, the strength and quality of steam-cured concrete at 24 hours will be greatly affected. Since the whole reaction of portland cement and water is accelerated with steam curing, every phase of the operation is more critical. This fact cannot be overemphasized. Experience in the field covering all types of concrete, (prestressed, pipe, slab and blocks) has shown that uniform 24-hour strength can only be attained if all these eight factors are closely controlled.

B. GENERAL INFORMATION

1. Admixtures

The use of certain admixtures will increase the quality of steam-cured concrete, and also speed production.

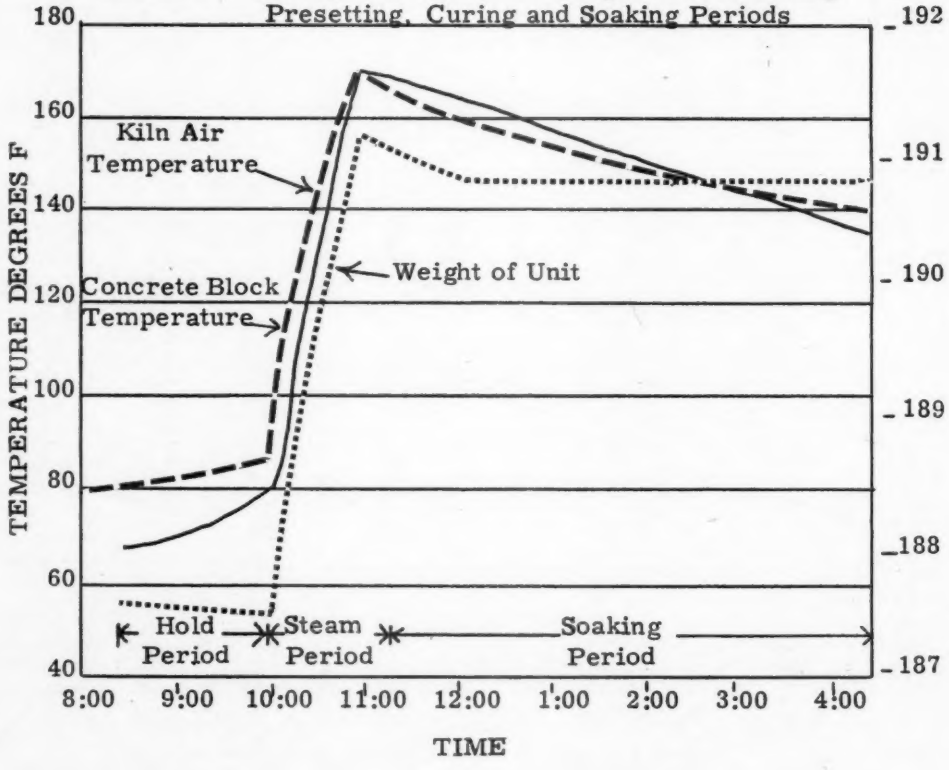
Plastiment—Although Plastiment is a retarder, Plastiment concrete will reach 1200 to 1500 psi at the same time as plain concrete with moist curing or steam curing. After this point, the Plastiment concrete will have higher strength than plain concrete. The 24-hour strength of steam-cured Plastiment concrete is invariably higher than equivalent concrete containing no Plastiment.

Four fluid ounces of Plastiment Liquid or one pound of Plastiment Powder should be used with steam curing, regardless of mix or curing temperature.

Calcium Chloride—Calcium chloride will accelerate setting time of concrete. Therefore, when using calcium chloride with steam-cured concrete a shorter hold period is required. Calcium chloride will be of benefit where very short steam curing cycles are employed. It will provide only nominal increases in strength with steam periods longer than 9 to 10 hours. Calcium chloride should be used only with the approval of the Engineer.

Air-Entraining Agents—Air-entraining agents do not affect the setting time of concrete. Where required for frost resistance, air entrainment may be employed. A decrease in strength may result from the use

FIGURE 3 - Curing Conditions and Weight of Unit During Presetting, Curing and Soaking Periods



This graph was taken from the paper "Curing Most Efficient Methods Determined by Tests" by George A. Mansfield, Huron Portland Cement Company.

of air-entraining agents.

2. Type of Cement

Concrete made from all five standard types of cement may be steam cured, However, type II, IV, and V, cement will set slower than type I and III, and will require a longer hold period. The 24-hour strengths when using type II, IV, and V cements will be lower than type III or type I cements, which gain strength faster.

3. Concrete Mix Design

Concrete mixes should be designed to obtain the 28-day strengths desired. Mix design should incorporate the admixture to be used in production. With optimum steam curing conditions, the 28-day strength can be approached in 24 hours.

4. Concrete Aggregates

All types of aggregates, regular and lightweight, may be used with steam curing, since steam curing operations only speed hydration of the portland cement in the concrete mix.

5. Equilibrium Temperature

The equilibrium temperature is the temperature at which both the concrete and the kiln atmosphere have the same temperature. As steaming starts, the concrete is cooler than the kiln atmosphere. This differential in temperature causes steam to condense on the concrete, and the concrete will gain weight (see figure 3). When concrete reaches the same temperature as the kiln atmosphere, no more steam will condense on it. As steaming is continued, to maintain but not increase the steam curing temperature, the vapor pressure of the water in the concrete may become greater than the vapor pressure of the kiln atmosphere. When this happens, water will evaporate from the concrete, causing drying shrinkage and a reduction in the rate of strength gain.

6. Test Cylinder Strengths

Since the rate of temperature rise of the concrete will be influenced by its mass, a standard 6" x 12" test cylinder will not necessarily indicate the strength of a large concrete girder steam cured next to it. Having a small mass, the concrete test cylinder will exhibit a temperature rise almost equal to that of the kiln. The temperature rise of a large concrete girder will be very slow and will lag behind the temperature rise of the kiln. As accelerated hydration takes place in the large concrete shape the internal temperature of the large mass of concrete will exceed that of the kiln due to heat of hydration. The heat of hydration generated in the small concrete test cylinder will be dissipated into the kiln air as it is generated and the small concrete test cylinder will usually remain at the kiln temperature. This is the reason why so often steam-cured cylinders do not reach the strength of large masses of concrete steam cured in the same kiln.

C. EQUIPMENT REQUIRED

The following inexpensive equipment and instruments are necessary to control operations so that consistent high strength and quality at early ages can be obtained with steam curing operations.

1. Standard slump cone or Kelly Ball—To measure concrete consistency as an indication of uniformity of concrete composition.

2. Concrete thermometer—A concrete thermometer is necessary to measure the temperature of the concrete mix which will determine how long a delay period will be required before steam curing.

3. Recording thermometer—A recording thermometer should be installed for every 100 feet of prestressing bed or kiln. A recording thermometer will record temperature during the preset period, rate of temperature rise and kiln temperature throughout the curing cycle. Good recording thermometers vary in price from $49.50 to $135.00. They are manufactured by Auto Lite, Weksler and Bristol, and others.

4. Automatic control valves—Control valves can be activated by the recording thermometer or by separate temperature switches. These automatic valves will control rate of temperature rise and maximum steam curing temperature according to a preset plan. They are manufactured by Minneapolis-Honeywell, Inc., and others.

5. Metal cylinder molds—Some types of cardboard molds will deform due to melting of the wax impregnation at steam curing temperatures. Metal molds are therefore preferred.

6. Schmidt concrete test hammer—Very often, due to the differences in mass, the concrete test cylinder will not indicate the strength of a large concrete member. The Schmidt impact hammer should be calibrated by testing cylinders of the same mix used in production with both the Schmidt hammer and a testing machine. When calibrated, the Schmidt hammer will determine the strength of concrete members with an accuracy of $\pm 5\%$. This hammer is distributed in the United States by Arthur R. Anderson of Tacoma, Washington, and priced at $135.00.

7. Fowler Vibra tachometer—Girders and other complicated shapes may exhibit pock marks and other defects due to insufficient or improperly spaced vibrators. The amplitude and frequency of vibration transmitted to the concrete or forms can be measured with the Vibra tachometer. This instrument can be used to set up a vibration pattern that will eliminate null points or areas of insufficient vibration. The Vibra tachometer is manufactured and sold by Viber Company, Burbank, California, and is priced at approximately $5.00.

D. RECOMMENDED PRACTICE

The following recommendations should be followed for optimum results. However, due to the size of concrete members, the shape of the prestressing bed or kiln, or other factors, it is sometimes not possible to

adhere strictly to these recommendations. However, every attempt should be made to conform to these tested methods.

1. Use a well-designed mix which has the proper workability and which will give the strengths desired.

2. Add 4 fluid ounces of Plastiment Liquid or 1 pound of Plastiment Powder per sack of cement to increase concrete quality and to speed production. Add both an air-entraining agent and Plastiment to concrete which will be exposed to freezing and thawing in service.

3. Experiment to determine proper hold period and kiln temperature required to obtain maximum strength for the particular mix.

4. Once a satisfactory steam curing schedule is established, control operations carefully to maintain desired results.

5. Supply sufficient steam to maintain a high humidity in the kiln atmosphere.

6. Do not allow steam to impinge directly on concrete or forms.

7. Design kiln, portable cover, or tarpaulin supports in such a manner that there is free passage of steam around the concrete.

8. Use metal test cylinder molds.

9. Place test cylinders in such a position that steam does not impinge directly on the cylinder molds.

10. Where the differential in mass between the test cylinder and the shape is such that the concrete cylinder does not indicate the actual strength of the large concrete mass, calibrate a Schmidt hammer with test cylinders of the same mix design, then test actual concrete members with the Schmidt hammer.

E. SPECIFICATIONS

At the option of the contractor, steam curing may be employed in lieu of moist curing. Where steam curing is employed, all standard specifications governing concrete composition and placing shall apply (note). In addition, the following specifications for steam curing shall apply.

1. The concrete mix temperature at the time of placing shall be maintained between 50 and 90°F. However, if heated aggregates or water are employed, the maximum mix temperature shall not exceed 70°F.

2. After mixing concrete, a preset or hold period of at least 2 hours and not more than 6 hours shall be observed before application of steam. During this period, the concrete temperature shall be maintained above 50°F.

3. Steam shall be introduced to the kiln enclosure at such a rate that the maximum temperature rise shall be less than 1°F per minute.

4. The maximum kiln temperature shall be 180°F. Steam shall be applied for a maximum of 30 hours.

5. The kiln or enclosure used to steam cure concrete shapes shall be so designed that there is no appreciable leakage of cold air into the kiln. Temperature throughout the kiln shall be maintained within ±5°F. The kiln or cover employed shall be such that there is free passage of steam around both sides and top of the shape being cured. Steam jets shall be set in such a manner that steam does not impinge directly on forms or concrete surfaces.

6. After steam curing, the concrete shapes shall be cooled slowly to prevent thermal shock to the concrete.

7. Test cylinders shall be cast in metal molds and covered with metal, glass, or other non-porous plates. They shall be placed in the kiln in such a position that they are exposed to the average kiln atmosphere.

NOTE: When employing steam curing, 4 fluid ounces of Plastiment Liquid or 1 pound of Plastiment Powder per sack of cement shall be used at the option of the contractor, when mix temperatures are between 50° and 65°F, the proportion may be lowered to 3 fluid ounces liquid or ¾ pound powder Plastiment.

8. The following records shall be submitted by the contractor:

a. Kiln temperature charts recorded by recording thermometers from time of placing concrete until completion of steam curing. One recording thermometer shall be placed in every 100 foot length of kiln.

b. Concrete mix temperature on each mixer load of concrete.

c. Concrete consistency on each mixer load of concrete shall be measured by a standard slump cone or Kelly Ball.

d. Compressive strength of test cylinders before application of prestress and at 28 days. Average of three cylinders shall constitute a test. Tests shall be reported on each half day production, or for every 50 yards of concrete.

e. Compressive strength of each concrete member as determined by a Schmidt impact hammer. Impact readings shall be made on vertical surfaces. An average of 15 impact readings shall be reported for each test. Three tests shall be made on each beam.

F. BIBLIOGRAPHY

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Graphs used in this discussion were taken from the above publications.

by M. H. YUAN

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PRESTRESSED CONCRETE IN TAIWAN

Prestressed concrete has fully gained its recognition in Free China after a brief period of research and successful experiment through the untiring efforts of some interested Chinese engineers. There are at present more than 10 projects using this new type of material in their construction. The performance has been invariably found highly satisfactory. The cheaper cost, finer appearance, and shorter work-time it offers to construction provide attractions especially to insular Taiwan where constructions are often limited by shortage of cement and steel, the two major construction materials. Given further development, prestressed concrete has promised a bright future in this part of the world, as elsewhere.

EDITOR'S NOTE: This paper was to be presented at the World Conference on Prestressed Concrete. However, since it was too late to be included in the program it is published in the Journal as it would interest many of our readers.

Prestressed concrete has been used in Free China for just a brief period of three years. It was agreed among the engineers that this new type of concrete construction is one deserving promotion, followed by intensive research conducted through concerted efforts of experts. It was the K. S. Engineering Co. who first tried on the experiment. At present there are more than ten projects using prestressed concrete in their construction in Free China. Some have been completed, others are still under construction, but the results are all the same: highly satisfactory. This result has provided the Chinese engineers with a clear conception and basic understanding of the importance of prestressed concrete in the construction field.

Constructions in Taiwan, Free China, have taken a great stride in recent years. However, the production of cement in Taiwan is not enough to meet the local requirements. Steel is also depending upon foreign supply. Under such circumstances, many constructions are hampered from time

to time due to material limitations. In view of the material shortage, the Chinese Civil Engineers Association in 1954 set up a working group for the research and promotion of prestressed concrete, in which the writer serves as the convenor. The working group first conducted an extensive study on the quality and gradation of the aggregates available in the province. Then cylinder tests made with different proportions of all grades of aggregates were conducted. The results of these tests proved that the materials available in Taiwan can meet A. S. T. M. standards, thus assuring us that prestressed concrete made of such materials will meet rigid specifications. The K. S. Engineering Co. in doing the research and experimenting on prestressed concrete has kept pace with the program adopted by the Chinese Civil Engineers Association in producing high strength concrete.

In the spring of 1955, the Chinese prestressed concrete expert in Thailand, Mr. Y. C. Chu, returned to Taiwan on leave. With the cooperation of Mr. Chu, we designed a number of prestressed concrete structures, and began to manufacture them. After untiring effort of research, more than ten kinds of 12-meter beams, slabs, railroad ties, railway crossing slabs, etc., were made by pretensioned and post-tensioned methods. For post-tensioning, embedded anchorage, twisting and zigzag pull methods were used, while for pre-stressing, ordinary hydraulic jacking was used. More than three hundred Chinese and foreign engineers were invited to witness the load test of these products; the result was very satisfactory. Mr. Chu's method of pre-stressing requires no imported materials or foreign equipment; all the anchorages, plates and jacks for stressing can be made locally. After the test was found satisfactory, Mr. Chu's pre-stressing method (embedded anchorage) was granted patent by the Chinese government.

The first prestressed concrete structure built with Mr. Chu's method is the five-tracks railway bridge of the Pingtung Mill, Taiwan Sugar Corporation. This bridge has 10 beams of 18.4 meters. The design was completed in August 1955. At the same time the Taiwan Sugar Corporation prepared an alternative design with conventional concrete. Both designs were put to bidding at the same time. As a result, the competitive price for the design with prestressed concrete was 24% cheaper than the one with conventional concrete. So it was decided that the construction should be done with the new method. Each track of this bridge consists of two prestressed concrete T-beams. The prestressed T-beams were precast on open ground about 500 meters from the bridge site. After load testing, these beams were moved by rail to the bridge site and were erected on the existing abutment.

Figure 1 shows the completed structure.

Fig. 1—Railway bridge of Pingtung Sugar Factory

Fig. 2—First prestressed concrete bridge of Ta-Chi of Taiwan Highway Bureau.

Fig. 3—Tao-Wu Bridge of the Taiwan Highway Administration.

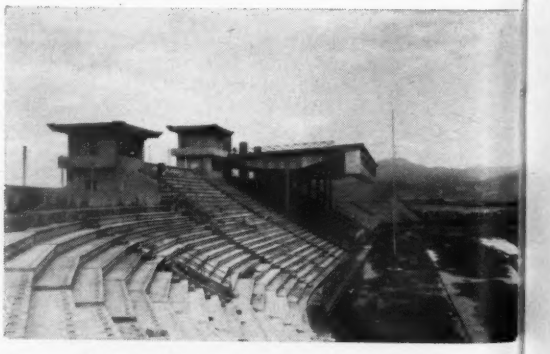


Fig. 4—Bleacher seats and roof slabs of Taipei Stadium.

At the same time we contacted S. T. U. P. Co. of France and in the winter of 1955 some Freyssinet jacks and Freyssinet cones were imported to Taiwan. The Freyssinet's method was adopted by the Taiwan Highway Bureau to design the first highway prestressed bridge. This bridge 30m long is designed for H-20. Four beams were cast in place. For each beam 18 strands of high tension wires were used, each wire strand consisting of 12 5mm wires. The design strength of the concrete was 4000 psi; all the test samples exceeded this minimum strength. This bridge is located across a deep valley, the bottom is about 18m below the bridge surface. Due to the rough terrain of the site, there was no suitable place for pre-casting and the only solution was to pour the concrete beams in place. In order to save false work materials, a concrete foundation was built on the river bed, the materials of two sets of Bailey truss were used as trestle bents, and the temporary deck was divided into three spans; i. e., 9m, 12m and 9m respectively. In order to reduce the load on the trestle, only one beam was poured at a time. This job was completed within 3 months and the performance was found very satisfactory. Furthermore, as this bridge is situated across a deep valley, shorter span length would require additional piers of 16-17m in height, if conventional concrete had been used. However, by using longer span prestressed concrete beams, a great deal of cost was saved.

Figure 2 shows the view of the bridge.

After two satisfactory performances mentioned above, prestressed concrete deeply impressed the Chinese engineers who began to recognize the advantages of this method of construction. Due to the attraction of low cost, high quality and fine appearance, there is a tendency that prestressed concrete structure will be widely adopted in various new constructions in the country. Since 1956, constructions that have used prestressed concrete are as follows:

(1) The roof structure of MAAG No. 63 club house:—The total area of the roofing is 684M², roof beam of 18m span by Freyssinet method, roof slabs of 4.50m span by pretensioning method. If steel truss was used in the roof structure, the cost would be approximately 35% higher than prestressed concrete, while if conventional reinforced concrete was used the cost would be 20% more. This construction was started in March and completed in June, 1955.

(2) Boiler tower of Sin-Ying Paper Factory, Taiwan Paper Corporation:—This is a construction of a 6-story tower for housing the boiler against wind or earthquake. Freyssinet method was used in the construction of beams and H columns. The floor slabs and roof slabs were cast by pretensioning method. This construction was started in September, 1956, and is still under construction.

(3) Tao-Wu Bridge of the Taiwan Highway Administration:—This bridge consists of 24-15m spans, (See Fig. 3). For each span, four pretensioned beams were used and erected by crane. The prestressed beams have a rail cross section, in which a total of 84-5mm steel wires were used. Four steel wires were used on the cross beams by Freyssinet method.

This construction was started in May, 1956 and completed in May, 1957. The use of prestressed concrete saved approximately 15% of the construction cost as compared with ordinary reinforced concrete.

(4) Taipei Municipal Stadium: (Fig. 4). As the stands had to be completed within 80 working days, the Chinese engineers used prestressed concrete in seat slabs and roofing slabs which were precast so that the work could be carried on simultaneously with the construction of the foundation and steel trusses. The 462 pieces of seat slabs and 72 pieces of roofing slabs were precast by pre-tensioned method. Within ten days after the work was started, four working platforms were made ready for use near the site. Each working platform provided space for construction of 12 slabs. Four days were required to complete a production circle, one day for the installation of forms, one day for pre-stressing and

Fig. 5—Railway bridge at Hsin-Hwa of Taiwan Sugar Corporation

Fig. 6—Taitung pineapple factory building of Taiwan Sugar Corporation

tying the steel bars, one day for pouring concrete and one day for steam curing and removal of the forms. All the seat slabs and roof slabs were manufactured in 48 days. Each seat slab, two tons in weight, was loaded on push carts by gallow frame, then pushed to the site and placed in position by derricks. The construction was started in April, and completed in August, 1956. This fully shows that the use of prestressed concrete can not only lower the construction cost, but also shorten the construction period.

(5) A bridge for the Shih-men Reservoir:—This is a highway bridge of three 30m spans. Twelve beams were pre-cast and prestressed by the Freyssinet method. They were moved to site after the sub-structure was completed. The work was started in October, 1956, and due to the delay of foundation work, the bridge is still under construction at present. If ordinary reinforced concrete were used for the construction, at least one more pier would have been necessary.

(6) Hsin-Hwa Railway Bridge of Taiwan Sugar Corporation: (Fig. 5). This bridge consists of four 20m spans. The beams were made by Freyssinet method. The work was started in August, completed and ready for traffic in November, 1956.

(7) The Taitung Pineapple Factory Building of Taiwan Sugar Corporation: (Fig. 6). This factory building is of a total area of 8, 160M^2 (40m x 204m). In the construction of the roof structure, girders, beams, and roof slabs, prestressed concrete was used. In making the girders, Mr. Chu's post-tensioned method was used, and in making the beams and roof slabs pre-tensioned method was used. In the design of the structure use of structural steel was also contemplated, however, after a comparison study was made prestressed concrete structure was found to be 20% cheaper. Besides, prestressed concrete structure is free from acid corrosion, and requires no painting or other maintenance costs. It was therefore finally adopted. In making the beams and roof slabs, three working platforms were used. After completion of precasting work, these working platforms were used for the floor of the factory. This job was started in October, 1956, and completed in May, 1957.

(8) Cover slab of Liu Kung Creek, Taiwan Highway Bureau:—When the highway at the southeastern suburb of Taipei city was widened, the Liu Kung Creek had to be covered up to serve as road surface. Prestressed beams of 6.52m span were made by pre-tensioned method, and laid crosswise. For the longitudinal tension, five groups of steel wires were used and the Freyssinet method was applied. The job was started in July and completed in November, 1956.

(9) Chu-nan factory building, Lungyi Paper Co.:—The girder used in this construction is of 21.1m span, pre-cast and pre-stressed by Freyssinet method and erected by crane. The beams, 6m long, and the roofing slabs 3m long were all pretensioned. The job was started in November, 1956 and is still under construction.

(10) Tunnel supports for various coal mines: (Fig. 7). Originally lumber was used in all the tunnel supports of coal mines in Taiwan, but due to the shortage of lumber supply and price increase in recent years, prestressed concrete supports which have longer life and can easily be handled have been considered very suitable for the purpose. Five hundred sets of such supports

Fig. 7—Tunnel supports for coal mines.

Fig. 8—Precast, prestressed concrete hollow piles for the Tanhsui River bridge.

and beams have been made and installed in various coal mines in Taiwan. Tested performance proved very satisfactory. Mass production of such supports will start this year.

(11) Pintung Victory Bridge of Taiwan Water Conservancy Bureau:—Ninety-three prestressed beams are required for this bridge, each 6.9m in length. They were precast and pre-tensioned and then shipped to the site for erection. This work was started in March and completed in May, 1957.

(12) Taipei Tanhsui River Bridge:—This bridge in 1055m long and 14.50m wide, it is an important link of the north-south communication system in Taiwan. There is a steel truss bridge which, over-aged, is incapable of sustaining the increasing heavy traffic; therefore, the new bridge is being constructed at a place one kilometer up stream of the old bridge. After more than one year's survey, boring, testing and designing, the Chinese bridge engineers were convinced that the use of structural steel would necessitate the importation of steel materials at higher cost and require longer time; and by using conventional concrete, more piers would be required, resulting in a higher construction cost. Adoption of prestressed concrete for this construction would mean much saving in time and cost. The job was started in December, 1956. Precast beams for five spans of 40m each and one span of 30m were constructed by the Freyssinet method, and beams for 55 spans of 15m each were constructed by pretensioned method. The design loading of the bridge is H-20, and the ultimate strength of the prestressed concrete is 300 Kg/cm^2. All the 380 beams and 648 prestressed concrete piles were precast and shipped to the bridge site. It is expected that the whole bridge will be completed at the end of this year. After completion, this will be the longest prestressed concrete bridge in the Far East.

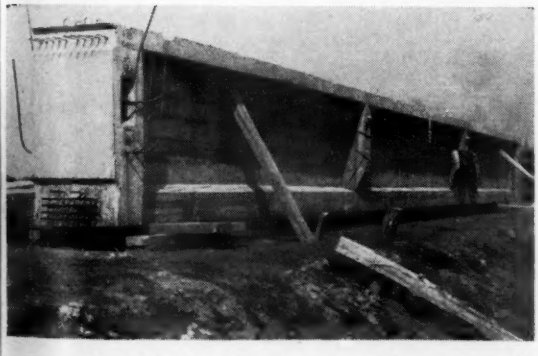


Fig. 9—Pretensioned 15-meter girders for Tanhsui bridge.

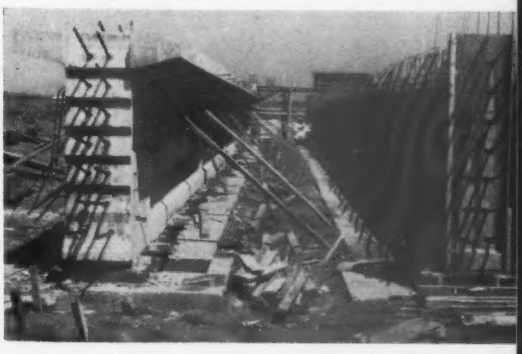


Fig. 10—Post-tensioned 40-meter girders for Tanhsui bridge.

Fig. 8 shows the prestressed concrete piles. Fig. 9 shows the 15m prestressed concrete beam. Fig. 10 shows the 40m prestressed concrete beam.

From the above information, it can be seen that Free China has made a good start in prestressed concrete construction and is going ahead with it by leaps and bounds. The Chinese Engineers are losing no time in research for its improvement with a view to fully realizing its technical and economical value. Within a brief period of three years, notable contributions have been made in prestressed concrete construction in our country through our untiring efforts. We are confident of our further achievement in the future and we assure you that we shall endeavor to contribute more.

The North Approach to the

BY K. DOUGLASS

A. B. Sanderson & Co., Ltd., Consulting Engineers

Vancouver, British Columbia

Erection of 120 ft. stringers on Second Narrows Bridge at Vancouver, B. C.

Second Narrows Bridge

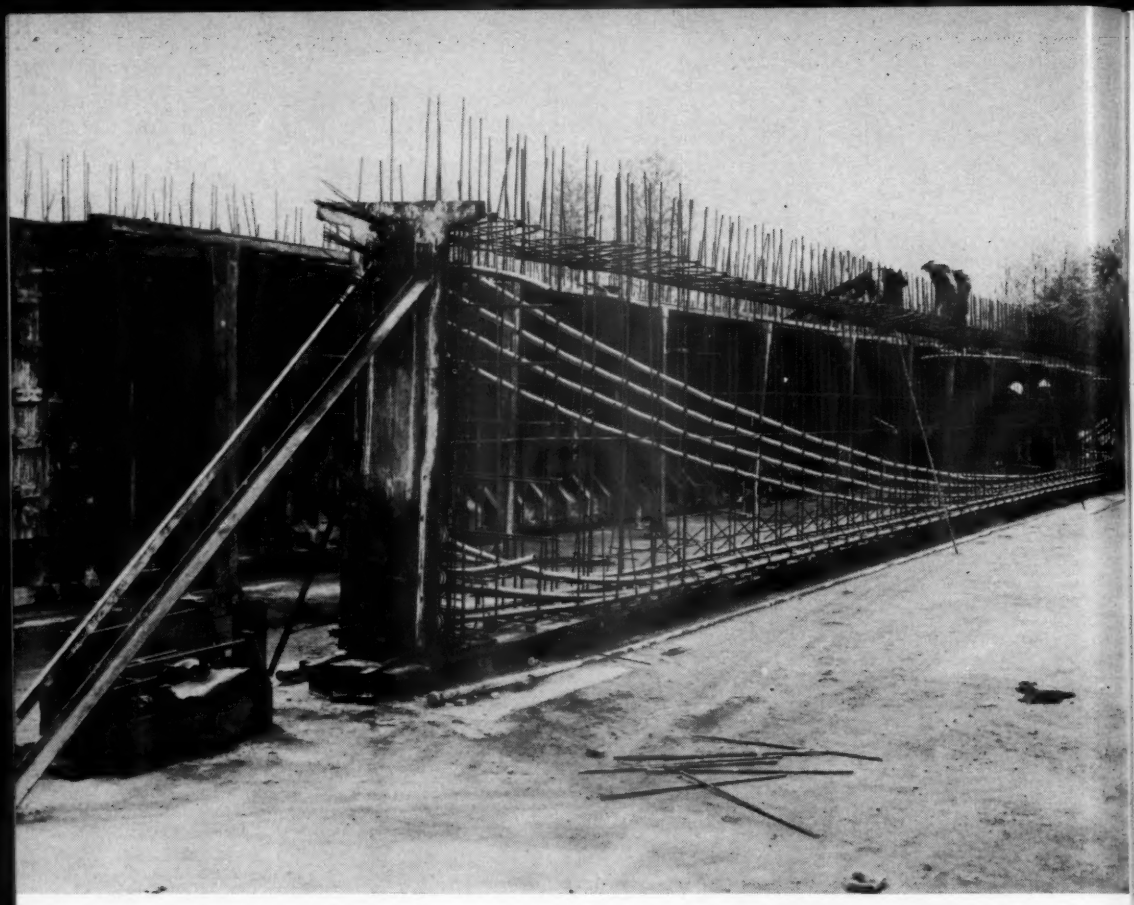
In November 1955 the Toll Bridge Authority of the Government of British Columbia called tenders for the construction of the substructure and approaches of a large bridge to cross the Second Narrows of Burrard Inlet adjacent to the City of Vancouver. In the tender they provided for alternative bids on 1,080 feet of approach at the north end of the bridge, 9 spans each 120 feet long, setting specifications for steel and for prestressed concrete. The firms of Peter Kiewit and Sons and Raymond International Limited, working on a joint tender, asked the firm of A. B. Sanderson and Company Ltd., consulting engineers, to prepare a preliminary design and estimates in prestressed concrete. After the contractors had been awarded the job they formed the operating company, Kiewit-Raymond Joint Venture and asked A. B. Sanderson and Company Ltd. to proceed with the final design and planning of the prestressed work.

DESIGN

The geometry of the north approach complicated the design problems considerably. The northern three spans are on a curve with different superelevations on the two halves of the six-lane deck. The longitudinal gradient of the approach is 5% with the height of the approach above ground varying from 52 feet to 106 feet. All piers are parallel so that the stringers throughout the job are the same length.

In order to provide for superelevation on the curve it was necessary to vary the deck thickness from 7½ inches to 11 inches and to provide stepped piers so that adjacent stringers are not on the same gradient. In short it was a problem of fitting straight sections to a complex curved surface.

It was decided finally to divide the 80-foot width of deck into two structurally independent halves, partly because the two



halves were not in the same plane, and partly because transverse expansion and contraction in the 80-foot width might lead to trouble in the bearings.

Specifications for the work are substantially the same as those prepared by the Bureau of Public Roads. The design is based on the A.A.S.H.O. code with no transverse distribution of loading, each stringer being designed to carry s/5 wheel-loads, where s is the spacing of the stringers in feet. The 4-stringer system and 6-stringer system proved to be most efficient and both systems received careful consideration. Finally the 6-stringer system was chosen on the basis of economy and size limitations. After considering a number of combinations of depth and stressing force the stringer was designed 7 feet deep with 10 cables, each consisting of 12 wires, 7 mm. in diameter (See Fig. 1). At the Contractor's request all cables were carried through to the ends of the stringers, and the helix was omitted from the cables. The Freyssinet system of prestressing was used throughout the job.

To avoid any possibility of structural deterioration of the transverse diaphragms the designers decided to stress two cables in the diaphragms. Subsequently they decided to stress the deck transversely, partly because of the cantilevered sidewalk brackets which could not readily be supported from the deck without stressing. Precast sidewalk panels set on the brackets completed the cantilever portion of the structure. In all transverse cables only 10 wires, 7 mm. in diameter are used because the spacing of deck cables is dictated by the module of the fencing.

PLANNING OF CASTING PROCEDURE

Fortunately the job schedule allowed ample time for the planning and detailing of the casting procedure. First the methods of handling were decided, based on the location and nature of the casting yard, a level area of irregular shape directly adjacent to the approach. The two carriers or gantries used throughout the job were then designed and manufactured. An interesting feature of the carriers is that the wheels used to carry them are retired bomber wheels from B 47's and B 52's (Fig. 2). The carriers are designed to be towed, and the 40-ton lifting jack, the power steering and the hydraulic emergency brake are operated through a single hydraulic pump.

The second problem was the number and detail of the casting beds. Concerning the number, it appeared that four would be

Figure 1

Figure 2

required for casting one beam each day, using steam curing and stressing the first four cables at a concrete strength of 3000 psi. To allow the steelworkers to fabricate their reinforcing cages on the bed called for another three beds, so a total of 8 was provided. Lubrited bearing plates from the bridge seats were built into the ends of the casting beds so that a beam would be properly supported when the first stage of stressing was completed. For the same reason the ends of the beds were built on timber piles driven into the alluvial soil. The system of locating the cables during the fabrication of the reinforcing cage is shown in Figure 3. Here a set of stanchions set in sockets beside each bed positions the cables exactly until the reinforcing cage was assembled.

The pouring was done by using the conveyor shown in Figure 4. An agitator truck hooked onto the conveyor carriage and pulled it along the form as required, so that the concrete could be distributed evenly on an inclined face. Four heavy external vibrators connected to the steel form and two internal vibrators provided the vibration.

The steel form was chosen rather than timber forms because of the problem of placing the concrete and the difficulty of using external vibration on heavy timber forms. Diaphragm stubs cast on the beams were at different angles because of the curve. Four sets of separate diaphragm boxes were required which were separated from the large panels by wedge-pieces to insure that the panels did not jam between diaphragm stubs. At the bottom the form was designed to fall free to the deck along the vertical side of the casting bed.

Finally the planning involved a number of details such as trucks to carry jacking equipment and grouting equipment, the assembly of cables in the metal sheath, and so on. The Freyssinet Corporation assisted with a number of details, such as the devices for holding anchorages against the end-forms. In general the time spent in planning was very profitable because the schemes and equipment developed in that time required little or no modification during the casting schedule.

MANUFACTURING SCHEDULE AND PROBLEMS CONNECTED THERETO

The plan called for one beam cast each day, five days a week. After the first beam was cast the crew spent a day in fitting the form for the second pour, but then hit the pace and maintained it until the 92nd beam was poured, working through some bitterly cold weather with temperatures down to 8°F. Finally the schedule was broken by a jurisdictional dispute between two union locals. Later the schedule was picked up and maintained until the end of the work.

Early in the schedule the working day began at 7 A. M. and ended as late as 7 P. M., but by the time the 6th beam was cast the work was completed in a regular working day.

The first work was restripping the form, an operation that went very rapidly. Setting up the form for the day's pour took until noon, with pouring directly after noon. Pouring took from 3 to 3½ hours leaving time to cover the beam for steaming before quitting time. While the form was being assembled another crew of 4 men stressed

Figure 3

4 cables in a beam 3 days old and in the afternoon the handling crew removed the beam to yard storage and braced it securely on heavy timbers. As the work progressed the stressing crew filled in the rest of the day with the second stage of stressing in the yard, and finally the grouting crew pressure-grouted the beams and covered the anchorages at the ends of the beams.

A problem developed in connection with the pouring. The two-inch slump recommended gave a strength of about 5800 psi at 28 days, a reasonable margin over the specified 5000 psi, but did not allow for careless placing. On some occasions the crew left the conveyor too long in one place, the concrete built up faster than the vibrators could handle it, and finally overfell on the front face, trapping a pocket of honey-comb. Trials with softer mixes were not successful because the sustained vibration tended to cause segregation, so finally the Contractor returned to a harsh mix with about 43% sand, 57% of ¾" gravel, and applied the necessary care in placing the mix. The resulting beams were excellent.

Stressing offered few problems after the schedule was established. In general the extension was calculated by using Freyssinet tables. A period of slipping wires ended with the thorough cleaning of all wire ends with steel wool. Otherwise the stressing was singularly uneventful.

Grouting proved to be a difficult matter. At the outset the Contractor investigated various methods of grouting and decided that the system developed by Prepakt Company offered the greatest hope of successful placing and full protection. A grout rated at 4000 psi with a flow-cone reading of 20 seconds was finally used. Regardless of the measures taken to flush the duct clean, a certain amount of dirt and scale remained in the duct. Apparently the grout picked up the dirt and built up a small plug as it progressed so that some ducts had to be flushed back. The wires were cleaned by means of a rotary wire brush and special care taken to flush out the duct after pouring, but the problem could not be rectified by these measures alone. The draft of the A.S.C.E.-A.C.I. Specifications recommends 120 feet as a limit for grouting in one direction, and experience on the job indicates that the limit may be set too high.

ERECTION

At present the erection procedure is well underway, with the last span nearing com-

Figure 4

pletion. The first span was slow work, but on the second and third the sub-contractor gathered speed until he was placing four and five a day.

In general the construction procedure is to bring a stringer to the two erection cranes beside the bridge, lift the stringer above the piers, and then walk the cranes across the bridge to place the stringer on its bearings. With minor variations this idea has been used throughout the erection job with success.

The stringers are tilted slightly on the piers and consequently have a tendency to deflect horizontally. To off-set this tendency the Contractor has had to tie the two halves of the deck together with wire rope and turnbuckles and will leave the ties embedded in the deck. Next he formed and poured the diaphragms between stringers using forms that were readily assembled and removed. The system of forming the deck between stringers is novel, at least in the Vancouver area. Corrugated sheet steel is suspended from the stringers by clips and will remain in place. Several sections of the deck have now been poured in this way and the Contractor is well pleased with the results. All that remains is to stress the transverse cables in the diaphragms and deck.

SUMMARY

On the Second Narrows Approach the yardage of concrete was great enough to warrant setting up a plant and the auxiliary equipment required for the large units. The Contractors did not try to cut any corners in setting up the plant, and the smooth operation can be attributed to their approach to the problem. Two members of the Raymond organization, J. D. Howell, the project manager, and John Guissinger, the project engineer, were responsible for the work.

Throughout the job the Contractors had a full measure of cooperation from the consulting engineers, Swan, Wooster and Partners, of Vancouver, and from their resident engineer, Carl Stanwick.

The success or failure of a job rests ultimately with the men in the field, and the creditable showing made on the Second Narrows Approach speaks well for the crew mentioned above.

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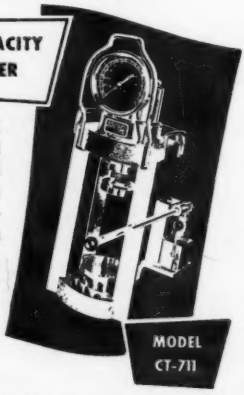

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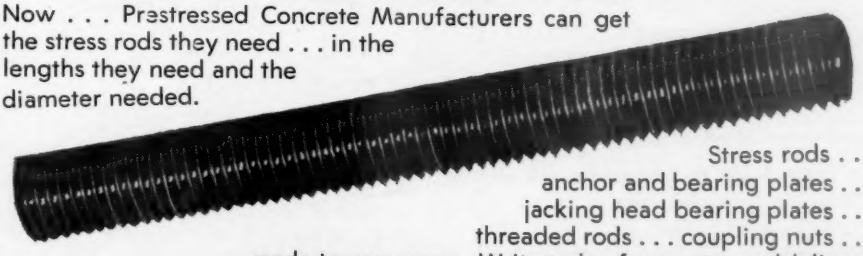

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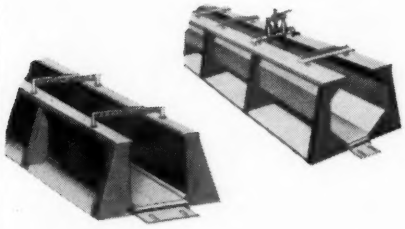

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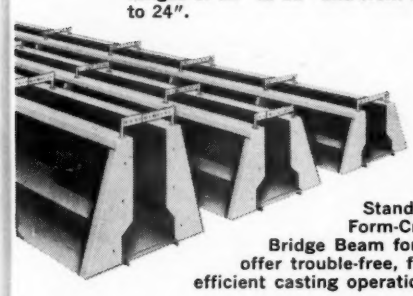

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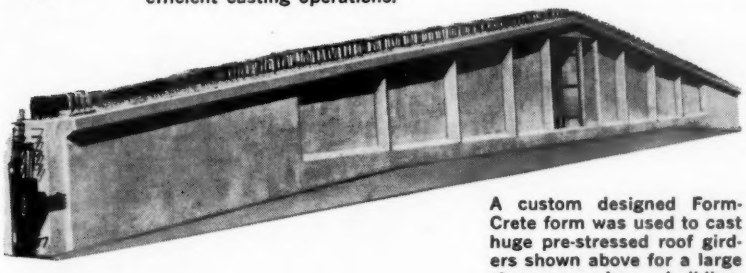

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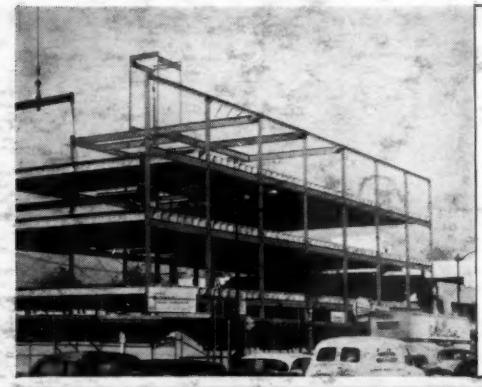
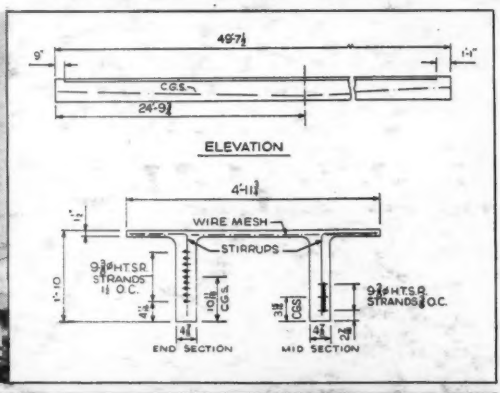
Prestressed concrete floor members with deflected strands were used in this Beverly Hills, California, 4-story office building. Steel plates cast in the concrete members were welded to the steel frame, so that the floor system serves as a diaphragm to stiffen the entire structure.

Each double-tee floor section is five feet wide x 22 in. deep and prestressed with 18 Roebling ⅜ in. diameter 7-wire uncoated stress-relieved prestressed concrete strands. A 2 in. poured-in-place slab on top of the precast section completes the structure. The 49-foot span is designed for 50 pounds per sq ft live load, plus 20 pounds per sq ft for walls and partitions.





Architects and Engineers
Cejay Parsons and Jack H. MacDonald

General Contractor
Jack H. MacDonald Co., Inc.

Prestressed Concrete Double-Tee Floor Sections
fabricated by
Rockwin Prestressed Concrete Corporation
Norwalk, California

Deflected Strands Increase Scope of Precast Prestressed Concrete

New technique permits longer spans with shallower and lighter beams; opens new areas for application of modern construction medium

Recently developed methods have added new impetus to the rapidly increasing use of precast pretensioned bonded prestressed concrete.

One of the most important of these new developments is deflection of the strands. This substantially increases the strength and bending resistance of a member without increasing its size.

A typical example is the 49 ft 7½ in. span floor system in the building illustrated above, using double-tee beams only 22 in. deep. Units for this span and load would have had to be much deeper and heavier if the prestressed strands were not deflected.

For data on tensioning elements and general information on prestressed concrete, write Construction Materials Division, John A. Roebling's Sons Corporation, Trenton 2, New Jersey.

Consult Roebling First . . . First in the U.S. with prestressing and tensioning elements

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